TRIAXIAL TESTING OF BITUMINOUS MIXTURES AT HIGH CONFINING PRESSURES

JUNE 1957

NO. 14

by

J.C. OPPENLANDER

Joint Highway Research Project

PURDUE UNIVERSITY
LAFAYETTE INDIANA

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

http://www.archive.org/details/triaxialtestingo00oppe

TRIAXIAL TESTING OF BITUMINOUS
MIXTURES AT HIGH CONFINING PRESSURES

TO: K. B. Woods, Director
Joint Highway Research Project

June 12, 1957

FROM: H. L. Michael, Assistant Director

File: 2-4-11
Project C-36-6K

Attached is a report entitled, "Triaxial Testing of Bituminous Mixtures at High Confining Pressures," by J. C. Oppenlander, a member of our staff. This study was performed under the direction of Prof. W. H. Goetz and was also used by Mr. Oppenlander as the thesis for partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

This project was a laboratory study that investigated the triaxial testing of bituminous mixtures at high confining pressures. The object of the research was to develop a better understanding of the basic factors that contribute to the development of shearing strength in bituminous aggregate mixtures.

The report is presented for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:hgb

Attachment:

cc: D. S. Berry
A. K. Branham
J. R. Cooper
W. L. Dolch
F. H. Green
J. T. Hallett
F. F. Havey
G. A. Hawkins
G. A. Leonards
W. H. Goetz
J. F. McLaughlin
H. L. Michael
R. D. Miles
R. E. Mills
B. H. Petty
C. E. Vogelgesang
J. L. Waling
J. E. Wilson
K. B. Woods
E. J. Yoder

FINAL REPORT

TRIAXIAL TESTING OF BITUMINOUS MIXTURES AT HIGH CONFINING PRESSURES

by

J. C. Oppenlander, Research Assistant

Joint Highway Research Project
Project C-36-6K
File: 2-4-11

Purdue University

Lafayette, Indiana

June 12, 1957

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to Professor W. H. Goetz, Research Engineer of the Joint Highway Research Project and Professor of Highway Engineering, Purdue University, for the generous time and effort expended in the guidance of this investigation.

Special thanks are extended to Professor G. A. Leonards, Associate Professor of Soil Mechanics, Purdue University, for his valuable suggestions.

The writer is grateful to the Joint Highway Research Project, Purdue University, for the materials, apparatus, special equipment, laboratory facilities, and part-time student help provided for this research project.

TABLE OF CONTENTS

	Page
LIST OF TABLES	v
LIST OF FIGURES	vii
ABSTRACT	xi
INTRODUCTION	1
REVIEW OF LITERATURE	6
Mohr's Theory of Strength	7
Triaxial Compression Test	9
Analysis of Triaxial Compression Test Data	14
MATERIALS	18
Mineral Aggregate	18
Bituminous Material	24
APPARATUS AND PROCEDURE	26
Preparation of the Aggregate	26
Preparation of the Test Specimen	28
Density Determinations	38
Triaxial Compression Test	42
Data Reduction	54
RESULTS	59
Open-Graded Mixture	61
Rational Triaxial Test	61
Specimen Height (Irrational Triaxial Test)	63
Type of Aggregate	64
Asphalt Content	65
Stabilometer Test	65
One-Size Mixture	69
Rational Triaxial Test	69
Density Determination	73
Design Criterion	75
Degree of Compaction and Specimen Height (Irrational Triaxial Test)	77
Type of Aggregate	86
Asphalt Content	86
Stabilometer Test	88
SUMMARY OF RESULTS AND CONCLUSIONS	93

Page

SUGGESTIONS FOR FURTHER RESEARCH 96

BIBLIOGRAPHY 99

APPENDIX A Derivation of Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing No Volume Change 104

APPENDIX B Derivation of Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction 107

APPENDIX C Triaxial Test Data 115

APPENDIX D Hveem Stabilometer Test 126

LIST OF TABLES

Table		Page
1.	Specific Gravities and Absorption of Aggregate	19
2.	Sieve Analysis of Open-Graded and One-Size Aggregates	22
3.	Physical Tests on 60-70 Penetration Grade Asphalt Cement	24
4.	Asphalt Contents for Different Aggregate Gradations	25
5.	Ratio of Normal Stress at 1.0, 5.0, and 10.0 Percent Strain to Normal Stress at 15.0 Percent Strain for Various Confining Pressure	78
6.	Triaxial Test Results, Open-Graded Mixture, Crushed Limestone, 10-in. High Specimen, Asphalt Content - 5.0%	116
7.	Triaxial Test Results, Open-Graded Mixture, Crushed Limestone, 4-in. High Specimen, Asphalt Content - 5.0%	117
8.	Triaxial Test Results, Open-Graded Mixture, Crushed Limestone, 10-in. High Specimen	118
9.	Triaxial Test Results, Open-Graded Mixture, Uncrushed Gravel and Natural Sand, 10-in. High Specimen	119
10.	Triaxial Test Results, One-Size Mixture, Crushed Limestone, 10-in. High Specimen, Asphalt Content - 5.0%	120
11.	Triaxial Test Results, One-Size Mixture (Vibrated), Crushed Limestone, 10-in. High Specimen, Asphalt Content - 5.0%	121
12.	Triaxial Test Results, One-Size Mixture, Crushed Limestone, 4-in. High Specimen, Asphalt Content - 5.0%	122
13.	Triaxial Test Results, One-Size Mixture (Vibrated), Crushed Limestone, 4-in. High Specimen, Asphalt Content - 5.0%	123

Table Page

14. Triaxial Test Results, One-Size Mixture, Crushed Limestone, 10-in. High Specimen 124

15. Triaxial Test Results, One-Size Mixture, Uncrushed Gravel and Natural Sand, 10-in. High Specimen 125

16. Stabilometer Test Results, Open-Graded Mixture, Crushed Limestone 129

17. Stabilometer Test Results, Open-Graded Mixture, Uncrushed Gravel and Natural Sand . . . 130

18. Stabilometer Test Results, One-Size Mixture, Crushed Limestone 131

19. Stabilometer Test Results, One-Size Mixture, Uncrushed Gravel and Natural Sand . . . 132

LIST OF FIGURES

Figure		Page
1.	Crushed Limestone Aggregate, Open-Graded and One-Size Gradations	20
2.	Uncrushed Gravel and Natural Sand Aggregate, Open-Graded and One-Size Gradations	21
3.	Gradation Curves for Open-Graded and One-Size Aggregates	23
4.	Cylindrical Mold and Typical Test Specimens; Triaxial, Modified Triaxial, and Stabilometer . .	30
5.	Pneumatic Vibrator with Compaction Foot and Specimen Mold	35
6.	Hydraulic Jack with Cylindrical Mold in Place for Double-Plunger Compaction	36
7.	Open-Graded and One-Size Mixture Specimens and One-Size Specimen Container	39
8.	Specimen Height Determination	41
9.	Equipment for Specimen Density Determination . .	43
10.	Data Sheet, Density Determinations - Bituminous Mixtures	44
11.	Rational Triaxial Compression Test	46
12.	Irrational Triaxial Compression Test	47
13.	Details of Triaxial Cell	48
14.	Preparation for the Triaxial Compression Test . .	52
15.	Data Sheet, Triaxial Test - Bituminous Mixtures .	55
16.	Typical Stress-Strain Curves	58
17.	Confining Pressure vs. Peak Normal Stress; Open-Graded Mixture, Crushed Limestone, Rational Triaxial Test, Irrational Triaxial Test	62
18.	Confining Pressure vs. Peak Normal Stress; Open-Graded Mixture, Rational Triaxial Test, Crushed Limestone, Uncrushed Gravel and Natural Sand	66

Figure Page

19. Normal Stress vs. Asphalt Content; Open-Graded Mixture, Rational Triaxial Test, Crushed Limestone 67

20. Normal Stress vs. Asphalt Content; Open-Graded Mixture, Rational Triaxial Test, Uncrushed Gravel and Natural Sand 68

21. Stabilometer Stability vs. Asphalt Content; Open-Graded Mixture, Crushed Limestone, Uncrushed Gravel and Natural Sand 70

22. Confining Pressure vs. Normal Stress at Various Percent Strain; One-Size Mixture, Rational Triaxial Test, Crushed Limestone 71

23. Normal Stress vs. Percent Strain; One-Size Mixture, Rational Triaxial Test, Crushed Limestone 76

24. Stress Ratio vs. Confining Pressure; One-Size Mixture, Rational Triaxial Test, Crushed Limestone 79

25. Confining Pressure vs. Normal Stress at Various Percent Strain; One-Size Mixture (Vibrated), Rational Triaxial Test, Crushed Limestone 81

26. Confining Pressure vs. Normal Stress at Various Percent Strain; One-Size Mixture, Irrational Triaxial Test, Crushed Limestone . . . 83

27. Confining Pressure vs. Normal Stress at Various Percent Strain; One-Size Mixture (Vibrated), Irrational Triaxial Test, Crushed Limestone 84

28. Confining Pressure vs. Normal Stress at 10.0 Percent Strain; One-Size Mixture, Rational Triaxial Test, Crushed Limestone, Uncrushed Gravel and Natural Sand 87

29. Normal Stress at 10.0 Percent Strain vs. Asphalt Content; One-Size Mixture, Rational Triaxial Test, Crushed Limestone 89

30. Normal Stress at 10.0 Percent Strain vs. Asphalt Content; One-Size Mixture, Rational Triaxial Test, Uncrushed Gravel and Natural Sand 90

Figure Page

31. Stabilometer Stability vs. Asphalt Content; One-Size Mixture, Crushed Limestone, Uncrushed Gravel and Natural Sand 91

32. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing No Volume Change; Open-Graded Aggregate Gradation, Specimen Height, 10.25 in. 105

33. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing No Volume Change; Open-Graded Aggregate Gradation, Specimen Height, 4.13 in. . 106

34. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 9.60 in., Range of C, 0.0 to 3.0 109

35. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 9.60 in., Range of C, 3.5 to 7.0 110

36. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 9.44 in., Range of C, 0.0 to 2.0 111

37. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 9.44 in., Range of C, 2.5 to 4.0 112

38. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 3.92 in., Range of C, 0.0 to 3.0 113

39. Relationship Between Change in Height and Cross-Sectional Area for Compression Specimens Experiencing Volume Reduction; One-Size Aggregate Gradation, Specimen Height, 3.80 in., Range of C, 0.0 to 2.0 114

Figure Page

40. Hveem Stabilometer Test 127

41. Data Sheet, Stabilometer Test - Bituminous Mixtures 128

ABSTRACT

Oppenlander, Joseph C. M.S.C.E., Purdue University, June 1957. Triaxial Testing of Bituminous Mixtures at High Confining Pressures. Major Professor: William H. Goetz.

This laboratory study investigated the triaxial testing of bituminous mixtures at high confining pressures. The object of this research was to develop a better understanding of the basic factors that contribute to the development of shearing strength in bituminous-aggregate mixtures.

The variables considered in this investigation were aggregate gradation, confining pressure, specimen height, degree of compaction, type of aggregate, and asphalt content. The two aggregate gradations used were referred to as open-graded and one-size and were incorporated with an asphalt cement (60-70 penetration) into bituminous mixtures. Both the open-graded and one-size mixtures perform adequately as paving mixtures for many service conditions, but only the open-graded mixture possesses adequate stability when evaluated by conventional testing methods. The variable of type of aggregate was measured by using an angular material, crushed limestone, and a comparatively round and smooth material, uncrushed gravel and natural sand. The asphalt content selected for these compacted specimens was that normally specified by the Indiana State Highway Department, and was maintained constant except for one phase of the study in which the variable of asphalt content was considered.

The triaxial compression test was selected to measure the shearing resistance of these bituminous-aggregate mixtures at a range of confining pressures from 0 to 150 psi. The rational triaxial test provided the only fundamental determination of the various factors that contribute to the stability of bituminous mixtures. The triaxial test specimens were compacted by the double-plunger method and by the double-plunger with vibration procedure, thus producing specimens at different density levels by varying the degree of compaction. Test specimens having a height to diameter ratio less than two were fabricated to measure the influence of the specimen height on the triaxial stability at high confining pressures. This phase of the research investigation was designated as the irrational triaxial test.

The laboratory results were presented in the form of graphical stress diagrams with the confining pressure plotted against the normal stress. For both the rational and the irrational triaxial tests, the open-graded mixture showed strength properties with a direct correlation to the Coulomb equation. Therefore, the fundamental strength properties of this mixture may be represented by the two constants, (c) cohesion and (ϕ) angle of internal friction. However, the shearing strength values produced by the irrational triaxial test were higher than the corresponding values for the rational triaxial test, and different values of cohesion and angle of internal friction were obtained by the two testing procedures.

The triaxial testing of the one-size mixture showed that low shearing strength was developed at low values of confining pressure and at small values of specimen deformation. With an increase in the confining pressure and/or the amount of specimen deformation, high values of shearing strength were obtained for this paving mixture. For the range of confining pressures investigated, approximately ninety percent of the shearing resistance was mobilized when the test specimen was deformed to a strain of 10.0 percent. The graphs of the confining pressure versus the normal stress at various strain values indicated a curvilinear relationship between these two values. Thus, the shearing strength of this one-size mixture cannot be ascertained wholly by such constants as the cohesion and the angle of internal friction, variables for this paving mixture. The complex interaction of lateral support, rearrangement of aggregate particles, and change in specimen volume prevented the exact evaluation of the effects of the specimen height on the triaxial stability of the one-size mixture.

For specimens made from both aggregate gradations, the crushed limestone aggregate produced higher triaxial stability values than the uncrushed gravel and natural sand for the range of confining pressures used. However, the selection of an asphalt content giving maximum strength for these two aggregate types in both the open-graded and the one-size mixtures was determined to be independent of the degree of the confining pressure.

With the open-graded mixture, positive correlation was obtained between the results of the rational triaxial test and the Hveem Stabilometer test for both the strength values and the selection of an asphalt content producing maximum strength. For the triaxial test values at 10.0 percent strain, satisfactory strength values were obtained, but the Stabilometer test produced stability values far below the acceptable minimum. Also, for maximum strength criteria, the rational triaxial test indicated that the asphalt content was near the value presently specified for construction, while a much lower asphalt content was determined from the Hveem Stabilometer test.

TRIAXIAL TESTING OF BITUMINOUS MIXTURES AT HIGH CONFINING PRESSURES

INTRODUCTION

The mechanical testing methods most utilized in the laboratory studies of the stability of bituminous paving mixtures can be listed as follows:

1. Hubbard-Field stability test.
2. Marshall stability test.
3. Direct compression test (ASTM version).
4. Hveem Stabilometer test.
5. Unconfined compression test.
6. Triaxial compression test.

Of these six testing methods, the triaxial compression test has proved to be an effective method for the study of the strength or the stability characteristics of bituminous paving mixtures. Stability may be defined as that property of a bituminous-aggregate mixture which permits the mixture to sustain the loads imposed by traffic without detrimental distortion or displacement. The data obtained from a triaxial test are usually analyzed in terms of the basic properties, the cohesion and the angle of internal friction, of a cohesive-granular mixture according to Mohr's theory of strength. This theory is thought to be more nearly in accordance with the behavior of bituminous mixtures than any of the other theories applicable to the strength properties

of a cohesive-granular material. The triaxial compression test provides a basic and rational approach to the evaluation of the fundamental strength properties of bituminous-aggregate mixtures, while the other methods listed provide only an empirical approximation to the expected stability of these paving mixtures.

From field observation, bituminous mixtures with both "one-size" and "open-graded" aggregate gradations appear to perform satisfactorily for many service conditions. For the purposes of this paper, a "one-size" aggregate is a material which passes the 3/4-in. sieve and is retained on the No. 4 sieve. An "open-graded" aggregate is one that is well graded from coarse to fine without any material passing the No. 200 mesh sieve. The results of conventional triaxial testing, on the other hand, show these one-size mixtures to be unstable under present design criteria, while the open-graded mixtures produce acceptable stability values. This illustrates that the present methods and theories of rational triaxial testing as applied to bituminous mixtures with a one-size aggregate fail to evaluate the true in-service stability properties of this paving mixture. A rational triaxial test is described as a triaxial compression test in which the diameter of the specimen is at least four times the maximum particle size, and the height of the specimen is at least two times its diameter. If the height to diameter ratio of the specimen is less than two, the triaxial compression test is defined as an irrational or a modified triaxial test.

Triaxial compression tests on bituminous-aggregate mixtures have been performed within a relatively low range of confining pressures (up to 60 psi), and the strength properties of these bituminous mixtures have been mainly predicated on the assumption that the Mohr rupture envelopes were straight lines. With the present-day tire pressures of 100 psi for trucks and 200 psi for airplanes and the probable future increase in these values, the lateral support that is provided the bituminous-aggregate mixtures is certainly in excess of the confining pressures, maximum of 30 psi, used in many triaxial tests performed for bituminous mix design.

The various factors that contribute to this lateral support of the bituminous-mixture surface of the flexible pavement are the tire pressure, the lateral support of the pavement adjacent to the loaded area, and the frictional resistance between the pavement and the tire and between the pavement and the base (29, 31, 34)[1]. If certain bituminous-aggregate mixtures are tested with a considerable range of confining pressures, the Mohr rupture envelopes may tend to curve. Depending upon the degree of this curvature, these non-isotropic mixtures may be analyzed by assuming either a straight or a curved Mohr rupture envelope. At present, there is not sufficient information available concerning the

---

1. Numbers in parentheses refer to references listed in the Bibliography.

rational triaxial testing of bituminous mixtures to ascertain which types of mixtures have straight line Mohr rupture envelopes and which ones have curved Mohr envelopes. The rational design of bituminous paving mixtures with curved Mohr envelopes has been developed theoretically by N. W. McLeod (32, 33).

Another important factor that is often neglected in the analysis of triaxial test data is the inability of the normal stress to reach a maximum value at a low percent of strain for a given confining pressure. Quite often a bituminous mixture will be judged unsatisfactory due to its low shearing strength at a small amount of strain, usually less than four percent, without proper consideration for the amount of shearing resistance that may be mobilized at strain values in the range of ten to fifteen percent. Any volume change and rearrangement of the aggregate particles that may occur are thus eliminated in the triaxial stability analysis of the bituminous paving material.

This is a report of an investigation of the triaxial stability properties of one-size and open-graded bituminous mixtures for a range of confining pressures up to 150 psi. The procedure consisted primarily of carefully testing laboratory-compacted specimens by the open-system triaxial compression test to determine their stability under the influence of different variables. The variable conditions introduced were aggregate gradation, confining pressure, specimen height, degree of compaction, type of aggregate,

and asphalt content. The variable of specimen height necessitated the molding of specimens whose height to diameter ratios were less than two. Therefore, this phase of the investigation is referred to as the irrational triaxial compression test study. The triaxial stability results were compared with the Hveem Stabilometer values for the two bituminous mixtures having the different gradations, but no effort was made to correlate the laboratory findings with the in-service performance that is affected by traffic and weather.

REVIEW OF LITERATURE

Many laboratory investigations have been conducted in an attempt to measure the strength or stability of compacted bituminous-aggregate mixtures and to develop empirical methods for the design of these mixtures and for the evaluation of their required thicknesses as the wearing surfaces of flexible pavements (37, 43). However, upon reviewing the many published articles written on these subjects, it is readily apparent that the established design procedures are only applicable to bituminous mixtures possessing a dense-graded aggregate. Since certain bituminous mixtures having a very open aggregate gradation cannot retain a given molded shape without the proper support, it is evident that this paving mixture must develop the required confining effects to sustain the applied traffic loads. In all the design procedures except the triaxial test method, the test specimens of these mixtures would possess a very low stability value because these methods do not provide sufficient confinement to this mixture. Excluding these empirical test methods, the triaxial compression test is the only suitable method for the fundamental evaluation of the true stability of this bituminous mixture having a one-size aggregate.

This review of literature is restricted to articles directly pertaining to the following subjects as related to the design of bituminous-aggregate mixtures:

1. Mohr's theory of strength.

2. Triaxial compression test.

3. Analysis of triaxial compression test data.

Mohr's Theory of Strength

A basic understanding of the strength properties of soils (a three-phase system of solid particles, water, and air) has been most accurately predicated by Mohr's theory of strength (23, 44, 45). Since a bituminous-aggregate mixture is a plastic material similar to soils and is a three-phase system of solid mineral aggregates, plastic bituminous material, and air, some bituminous paving engineers projected that the strength properties of these bituminous construction materials could be predicted by the same theory applicable to soils. This fundamental concept has been investigated, and it has been determined that the observed action of a bituminous-aggregate mixture under the action of a loading system is most nearly in accordance with Mohr's theory of strength than with any of the other strength theories (40). It may be concluded then that this analysis provides a basic and logical approach to the evaluation of the fundamental strength of bituminous-aggregate mixtures.

The stability of a bituminous-aggregate mixture (the ability to resist excessive deformation due to the application of a system of loads) is thought to be derived from the combination and the interaction of two fundamental and independent properties of the bituminous mixture. These properties are cohesion and internal friction.

The property of internal friction is that contribution to the shearing strength of the bituminous mixture derived from the frictional resistance to the sliding of one aggregate particle over another and from the interlocking of the particles. The second contribution to the shearing strength is termed cohesion which is considered to be mainly a function of the bituminous material. Cohesion results from the ability of the bituminous material to cement or bind the individual particles of mineral aggregate to resist the applied shearing forces (17). In regard to internal friction and cohesion, V. A. Endersby (10) stated that:

> Cohesive resistance is spoken of as "initial resistance" because it is independent of load, while frictional resistance increases with load. The two have to be combined to show the real load resistance of the material.

Also, Endersby (9) classified the stability factors due to the two components, aggregate and bituminous material, of a bituminous-aggregate mixture by the following outline:

1. Stability factors due to aggregate.
    - a. Internal friction.
    - b. Structural resistance.
    - c. Cohesion.
2. Stability factors due to asphalt.
    - a. Cohesion.
    - b. Viscous resistance - measured by L. W. Nijboer (38).
    - c. Apparent cohesion.
    - d. Other variables depending on the asphaltic material.

The present practice in the evaluation of triaxial test information on bituminous mixtures is predicated on the assumption that the Mohr rupture envelope is a straight line. Thus, constant values of the angle of internal friction and of cohesion are assigned to a given bituminous paving mixture for any given condition. However, many bituminous mixtures actually possess a Mohr rupture envelope that is curved. In discussing this phenomenon, Endersby (10) suggested that:

> In bituminous paving mixtures and aggregates the envelope is usually found to be curved, which leads to four possible theories as to the nature of the changes represented during loading. We may assume that friction decreases with load while cohesion is constant; that friction increases while cohesion decreases; that friction is constant while cohesion increases; or that a third factor, "structural resistance" akin to column or arching action (depending on dimensions of specimen) is present.

As Mohr's theory of strength has been extensively developed in many textbooks on soil mechanics (44, 45) and in numerous published articles dealing with soils and bituminous materials (1, 10, 12), any further discussion of this important concept, which is common knowledge to materials engineers, is unwarranted.

Triaxial Compression Test

The triaxial compression test has been defined by V. A. Endersby (10) in the following description:

> The word "triaxial" is applied to a form of mechanical test under which a load is applied axially to a cylindrical specimen, while a supporting pressure is maintained against its sides by water, air, or other means. The limiting form in one direction is the simple compression test in which the supporting pressure is zero. . . . In the other direction the limiting case is that of a load applied axially to a cylinder which is completely restrained laterally. The stress-resistant properties of the material tested triaxially are derived from the relation between the testing load and the supporting pressure.

There are several important reasons for employing the triaxial compression test to measure the stability properties of various bituminous mixtures. These are summarized in the following tabulation:

1. A fundamental measure of the true stability or shearing strength of the bituminous-aggregate mixture is provided.

2. The results of the triaxial compression test are directly applicable to analysis by Mohr's theory of strength, which is valid for cohesive-granular materials (40).

3. The application of a controlled and measured lateral support, which enables flexible pavements to sustain the large wheel loads and the tire pressures now being employed, are permissible only in the method of triaxial testing.

4. Only the triaxial compression test will properly measure the load or stress required to cause a certain amount of deformation (36).

Concerning the triaxial test as a rational approach to the evaluation of the strength of bituminous mixtures, N. W. McLeod (29, 30, 34), has written:

> When it comes to designing bituminous mixtures of any specified strength, the stability tests in most common use, Hubbard-Field, Marshall, and Hveem Stabilometer, are not able to measure the strength of bituminous mixtures in terms of shear or any other fundamental property on a pounds per square inch basis. They are strictly empirical tests. Empirical methods have a serious drawback in that it is dangerous to extrapolate their results to cover conditions beyond those under which they were established. With empirical methods also, it is difficult to avoid either overdesign or underdesign. In addition, in every engineering field there should be the ultimate objective of establishing rational methods of design, in which the strengths of all materials employed are utilized on a unit strength basis.

Other authors have also expressed the desirability of the triaxial test. The following is taken from an article by B. A. Vallerga (48):

> It has long been recognized by those best informed on the behavior of granular materials when subjected to highway loadings that the underlying principles embodied in the triaxial test provide the most promising approach to the problem of properly and accurately evaluating the ability of both soils and asphaltic mixtures to sustain heavy vehicle loads in the design of highway pavements.

There are two general types of triaxial compression tests. One type is known as the open or applied-pressure type, while the other method is classified as the closed or induced-pressure type (18). In the open-type, triaxial compression tests are performed on several different specimens with a different confining pressure for each specimen. There are many references to this type of test (7, 13, 14, 17, 27, 35). In the closed type of test, only one specimen is tested to obtain a relationship between the vertical stress and the induced lateral stress. The only procedure for bituminous

mixture design which utilizes the triaxial test is predicated on the closed-system method, this design procedure being known as the "Smith Triaxial Method of Mix Design." However, this design method has been used principally as a research tool (39, 46, 47).

For the purposes of this research investigation, the type of triaxial compression test utilized was the open-system method, as the apparatus for this procedure was readily available. Furthermore, this system permitted the confining pressures to be accurately controlled.

Some groups of paving engineers regard the Hveem Stabilometer as a form of the "closed" triaxial compression test. Recently, many laboratory investigations have been made to measure the stability properties of soils and bituminous mixtures with the Stabilometer (11, 21, 22, 25, 41, 48). If the Hveem Stabilometer test is to be considered as a triaxial test of the closed type, the following items must be appraised in the evaluation of the results obtained from this method of test:

1. A specimen having a height to diameter ratio less than two ($h/d = 0.625$ approximately) is specified. Thus, an unmeasured amount of lateral restraint is applied to the test specimen due to the shearing forces acting on the ends of the specimen.

2. The amount of confining pressure cannot be controlled, only measured.

3. The deformation of the specimen is not controlled in the test procedure. Some bituminous mixtures may require a certain degree of deformation before sufficient shearing resistance is mobilized.

4. The test procedure is quite empirical, when such items as the displacement pump operation, the coating of open-textured specimens with plaster of Paris, and the use of fixed loads are analyzed.

Certain empirical pavement design procedures have been developed for specifying the thicknesses of the various components of a flexible pavement, after the triaxial stability values of these individual components have been ascertained (4, 5, 8, 15, 20, 21, 24, 28, 30, 31, 40, 49). In the application of triaxial test information to the design of flexible pavements, L. E. McCarty (25) has presented the following limiting criteria:

> Thus, while it is not correct to base the design of comparatively thick base courses on results from a Hveem stabilometer test on the small Hveem specimen without applying a height correction derived experimentally or from theory, if possible, for the relatively great difference in structural strength, neither is it correct to apply uncorrected results from the test on a tall specimen in the design of thin bituminous-surface courses. In either case independent correlations with field performance would be required for the correct design of various thicknesses of pavement unless a suitable correction is applied to the original test results.

However, N. W. McLeod (29, 34) has developed a method of flexible pavement design in which rational triaxial specimens (height to diameter ratio is equal to two or greater) are tested in an open-triaxial-test system.

Analysis of Triaxial Compression Test Data

The data obtained from triaxial compression tests on specimens of compacted bituminous-aggregate mixtures may be evaluated in a variety of ways. As previously discussed, the shear failure of bituminous-aggregate mixtures is in close agreement with Mohr's theory of strength, and the values measured in the triaxial compression test are best analyzed according to this fundamental theory. Mohr's theory states that a cohesive-granular material will fail along a given shear plane when an optimum combination of normal stress and shearing stress is produced within the stressed specimen. The normal and the shearing stresses at any point on any plane can be determined either algebraically or graphically from the principal stresses at that point. These equations and the Mohr diagram are explained in considerable extent in most textbooks on soil mechanics (44, 45). When the triaxial test data are secured for plotting Mohr circles, these circles represent the combinations of the normal and the shearing stresses acting on the various planes within the stressed body. These different circles are drawn for the conditions of various confining pressures and their corresponding normal stresses.

The method most commonly utilized for the analysis of triaxial test data, is predicated on the assumption that the Mohr rupture envelope is a straight line and tangent to the Mohr circles representing the failure condition. This line

is then a linear representation of the shearing strength properties of the material tested. This straight-line Mohr rupture envelope is also a graph of the Coulomb equation for the shearing stress (3):

$$s = c + p \tan \phi$$

where s = shearing stress or strength

c = cohesion

p = normal stress

ϕ = angle of internal friction

Thus, the angle of internal friction and the cohesion, the basic properties of a cohesive-granular mixture, are evaluated as the slope and the shear axis intercept for zero normal stress, respectively, of a linear Mohr envelope.

This method of analysis has been employed extensively in the laboratory evaluation of various bituminous-aggregate mixtures. Some of the areas of investigation include the applicability of a vacuum triaxial technique (14), the comparison of the triaxial test method with the empirical Marshall Method of mix design (13), the effects of the aggregate shape on the stability of bituminous mixtures (17), the evaluation of the factors of aggregate orientation and the method of compaction (16), and the determination of the plastic properties for various bituminous mixtures (38). One investigator (6) has attempted to develop a simple shear test from which the Mohr diagram can be ascertained.

For certain bituminous mixtures, the Mohr rupture envelope appears to be curvilinear. Thus, the angle of

internal friction and the cohesion are not constant for a given evaluation, but rather they vary for different confining pressures. Depending upon the degree of this curvature, the bituminous mixture can be assumed as having either a straight or a curved Mohr rupture envelope. N. W. McLeod (32, 33) has devised a rational method for the design of bituminous mixtures with curved Mohr rupture envelopes, assuming that this curved line can be represented by an exponential equation. McLeod (33) has illustrated that:

> By employing the best straight Mohr envelope for a bituminous mixture for which the triaxial data actually plot as a curved Mohr envelope, the stability of the paving mixture might be overestimated by as much as 30 per cent.

A convenient method for analyzing triaxial test results, without plotting the Mohr diagram, is to plot a graph of the confining pressure versus the total normal pressure. This plot will yield a straight line if the Mohr rupture envelope is linear. W. M Aldous, R. C. Herner, and M. H. Price (1) have derived expressions for determining the angle of internal friction and the cohesion from the slope and the vertical intercept of this graph of the lateral and normal stresses. Although the utilization of these two equations are only applicable to isotropic bituminous mixtures having a linear Mohr rupture envelope (26), the plot of the confining pressure versus the total normal stress provides a convenient, graphical analysis of the strength properties of non-isotropic bituminous-aggregate mixtures (16) having curved Mohr rupture

envelopes. For these cases, the plots will be curved, and values of the angle of internal friction and cohesion will vary for different confining pressures.

In conclusion to this discussion, R. E. Gibson and W. S. Housel (12, 19) postulate that the angle of internal friction, as evaluated from the Mohr rupture envelope, does not actually represent the internal frictional resistance of a bituminous mixture; and that the cohesion, as determined from the vertical-axis intercept, does not correctly measure the shearing resistance provided by the bituminous material. It has been further pointed out that the Coulomb equation and the linear Mohr stress envelope violate the basic laws of static equilibrium when used with triaxial test data, as this equation, or its graphical representation, cannot be properly applied to the internal stability of granular materials (19).

MATERIALS

Since the type and the shape of mineral aggregates were considered as variables in this investigation, two locally-available mineral aggregates were selected for use in this study to ascertain the effects of these two variables on the stability of the compacted bituminous-aggregate specimens. A detailed description of these aggregates as well as the identifying test values for the selected bituminous material, the other basic component of the bituminous-aggregate mixture, are presented in the following two sections.

Mineral Aggregate

The mineral aggregates selected for this research project were obtained from two commercial aggregate companies, and their physical characteristics were not altered in the laboratory except for the gradation. Both the coarse and the fine aggregate fractions used in the individual bituminous mixtures were of the same aggregate type, either crushed limestone or uncrushed gravel and natural sand. The crushed limestone was obtained from the Ohio and Indiana Stone Company, Greencastle, Indiana, while the uncrushed gravel and natural sand were secured from the Western Indiana Sand and Gravel Company, Lafayette, Indiana.

The procedures outlined in the ASTM Standards (2) were followed in the determination of the specific gravities and

absorption values of these materials. Standard test method C127-42 applied to the coarse aggregate, while standard test method C128-42 was applicable to the fine aggregate fraction. The results of these tests are shown in Table 1.

TABLE 1

Specific Gravities and Absorption of Aggregate

Aggregate	Bulk Sp Gr	Apparent Sp Gr	% Absorption
Crushed Limestone	2.66	2.71	0.74
Uncrushed Gravel	2.60	2.74	2.00
Natural Sand	2.54	2.68	2.04

As previously mentioned, one of the variables in this investigation was the gradation of the aggregate. The open-graded aggregate corresponded to the requirements for the Hot Asphaltic Concrete Binder Course, and the one-size aggregate followed the Bituminous Coated Aggregate Surface (size No. 9 aggregate) gradation, as presented in the Indiana State Highway Department Specifications (42). Samples of these two gradations are pictured in Fig. 1 for the crushed limestone aggregate and in Fig. 2 for the uncrushed gravel and natural sand aggregate. The sieve analysis of these open-graded and one-size aggregates is presented in Table 2. A graphical plot of the gradation of these two aggregates is shown by the aggregate gradation curves in Fig. 3.

Fig. 1 Crushed Limestone Aggregate, Open-Graded and One-Size Gradations

Fig. 2 Uncrushed Gravel and Natural Sand Aggregate, Open-Graded and One-Size Gradations

TABLE 2

Sieve Analysis of Open-Graded and One-Size Aggregates

(Percent by Weight)

SIEVE		GRADING	
PASSING	RETAINED	OPEN	ONE-SIZE
--	3/4"	0	0
3/4"	1/2"	17.5	29.2
1/2"	3/8"	21.4	35.4
3/8"	#4	21.4	35.4
#4	#6	2.9	0
#6	#8	1.7	0
#8	#16	10.5	0
#16	#50	20.7	0
#50	#100	3.6	0
#100	#200	0.3	0
#200	--	0	0

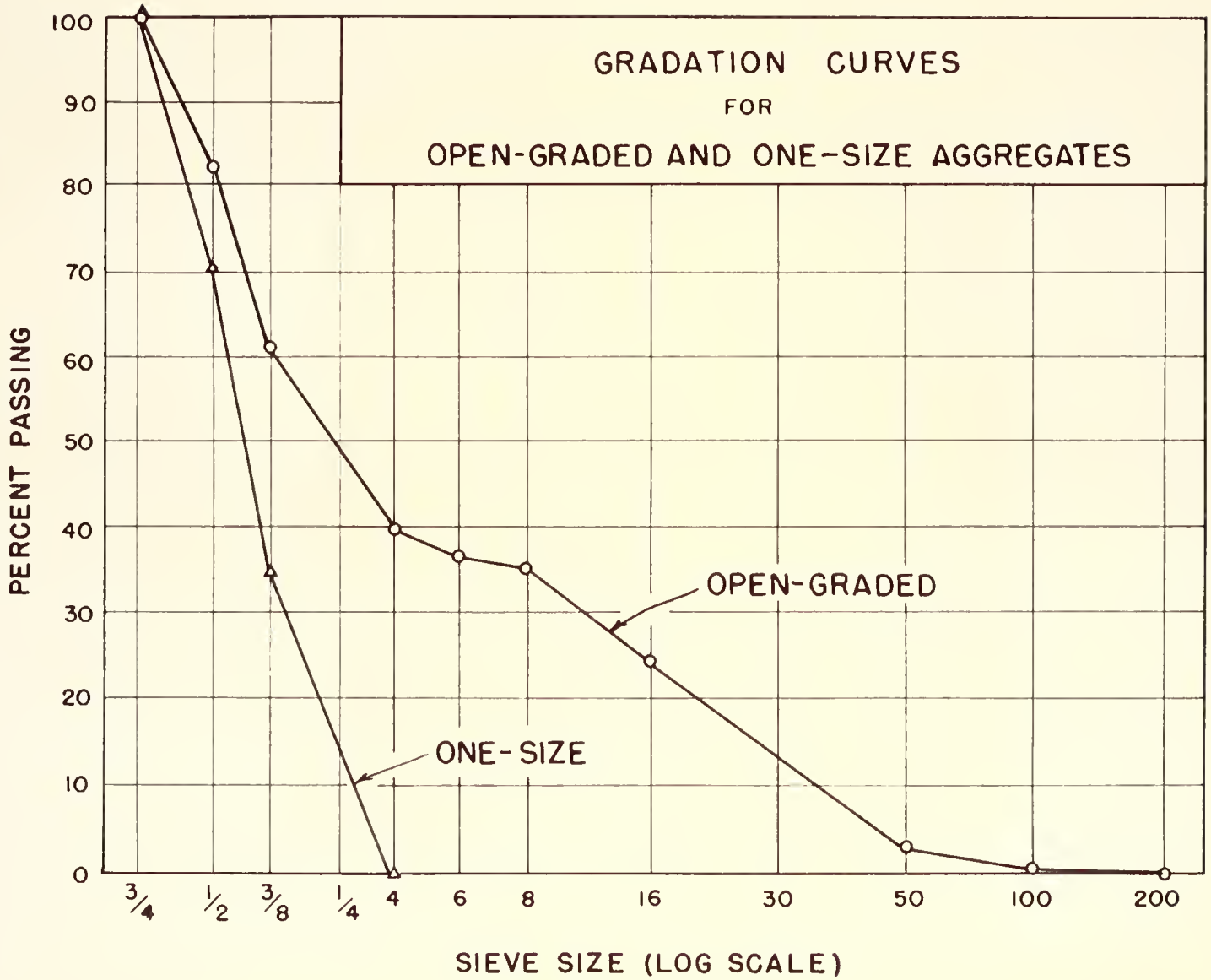

GRADATION CURVES
FOR
OPEN-GRADED AND ONE-SIZE AGGREGATES
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
OPEN-GRADED
ONE-SIZE
3/4
1/2
3/8
1/4
4
6
8
16
30
50
100
200
SIEVE SIZE (LOG SCALE)

FIG. 3

Bituminous Material

The bituminous material used in this study was a No. 65 Paving Cement obtained from the Texas Company, Port Neches, Texas. This asphalt cement is further classified as a 60-70 penetration grade. Several standard tests were performed in the laboratory on this asphaltic material. The results of these tests along with the designation of the applicable American Society of Testing Materials standard method of test are presented in Table 3.

TABLE 3

Physical Tests on 60-70 Penetration Grade Asphalt Cement

Test	ASTM Test No.	Results
Penetration - 1/100 cm (77 F, 100 g, 5 sec)	D5-52	66
Specific gravity (77 F / 77 F)	D70-52	1.018
Ductility - cm (77 F, 5 cm/sec)	D113-44	150 +
Solubility in CCl_4 - %	D165-42	99.81
Loss on heating - % (50 g, 5 hr, 325 F)	D6-39T	0.01

The asphalt contents for the different aggregate gradings were maintained at 5.0 percent by weight of the total mixture throughout most of this research investigation. These percentages of asphalt cement used in the open-graded

and the one-size aggregate mixtures were those normally specified for actual construction by the Indiana State Highway Department.

In one phase of this research study, the asphalt contents of the open-graded specimens and the one-size specimens were altered to furnish information concerning the effects of asphalt content on the triaxial stability values of these two differently graded specimens tested at a range of confining pressures. The various asphalt contents used in the open-graded and the one-size compacted bituminous-aggregate specimens are presented in Table 4.

TABLE 4

Asphalt Contents for Different Aggregate Gradations

Aggregate Gradation	Asphalt Content (Percent of total mixture weight)
Open-graded	4.0, 5.0, 6.0
One-size	3.0, 4.0, 5.0

APPARATUS AND PROCEDURE

The apparatus and the procedure utilized in this laboratory investigation will be discussed in the following five general operations:

1. Preparation of the aggregate.
2. Preparation of the test specimen.
3. Density determinations.
4. Triaxial compression test.
5. Data reduction.

Preparation of the Aggregate

The following apparatus were used in the preparation of the mineral aggregate:

1. Gilson mechanical sieving machine.
2. Tyler Company Ro-Tap Testing Sieve Shaker.
3. U. S. Standard sieves; sizes 3/4 in., 1/2 in., 3/8 in., Nos. 4, 6, 8, 16, 50, 100, and 200.
4. Torsion balance - capacity of 4.5 kg.

The mineral aggregates as received from the commercial sources consisted of both coarse aggregate and fine aggregate fractions. In order to provide a high degree of control over the aggregate gradations, these mineral aggregates were separated into various sieve size fractions and then were recombined in the desired proportions. These aggregates were allowed to dry in air before sieving.

The coarse aggregates were separated by a Gilson mechanical sieving machine to obtain the required aggregate fractions of 3/4 in. to 1/2 in., 1/2 in. to 3/8 in., and 3/8 in. to No. 4. The Tyler Company Ro-Tap Testing Sieve Shaker was used to divide the fine aggregates into the required aggregate fractions of No. 4 to No. 6, No. 6 to No. 8, No. 8 to No. 16, No. 16 to No. 50, No. 50 to No. 100, and No. 100 to No. 200. The separate fractions of the aggregate were stored in covered containers until needed. The sieving operations were carried on throughout the investigation so that a supply of aggregate was always available.

Just prior to the preparation of the test specimen, the separate aggregate fractions were combined into aggregate mixtures having the correct proportions by weight as given in Table 2. To produce a rational triaxial test specimen having a diameter of 4 in. and a height of 10 in., combined-aggregate weights of 4750 g and 3420 g were selected for the open-graded mixtures and the one-size mixtures, respectively. When the irrational triaxial test specimens having a diameter of 4 in. and a height of 4 in. were prepared, the aggregate weights were 1900 g for the open-graded mixtures and 1368 g for the one-size mixtures. For the taller specimens (10 in.) only one-half of the required aggregate needed for one specimen was weighed into a batch pan because of the limited capacity of the mechanical mixer. However, the total weight of aggregate used in the shorter specimens (4 in.) was batched into a single pan.

Preparation of the Test Specimen

In the preparation of the triaxial test specimens, the following apparatus were employed:

1. Peerless gas oven with temperature control.
2. Modified Hobart electric mixer with flat bottom mixing bowl.
3. Ohaus beam balance.
4. Assembled compaction mold.
5. Quart sauce pans.
6. Flat shallow pans.
7. Large metal spoons.
8. Armored thermometers, 50 to 450 F.
9. Stop watch.
10. Circular paper discs - diameter of 4 in.
11. Round steel bar - diameter of 3/4 in. and weight of 1.4 lb.
12. Blackhawk hydraulic compaction device - 50-ton capacity with three ranges.
13. Leather mallet.
14. Hobart Brothers air compressor.
15. Cleveland pneumatic vibrator with a compaction foot having a diameter of 2.5 in.
16. Sheet metal containers - diameter of 4 in. and height of 12 in.
17. Cold water bath.

Before the actual molding operations were started, the asphalt, the aggregate, the mixing equipment, and the compaction equipment were prepared for use. The bulk asphalt cement was obtained from the refinery in five gallon cans. Only the required amount of asphalt cement needed for one specimen was heated at one time. This was removed from the container by cutting with a heated, large metal spoon and was placed in a quart sauce pan. An additional forty grams of asphalt cement were placed in the sauce pan to compensate for that material adhering to the inside of the pan.

The steel cylindrical compaction mold having an inside diameter of 4.0 in. was cleaned and all faces that were to be in contact with the bituminous mixture were coated with a light lubricating oil. A view of the mold disassembled is shown in Fig. 4. The compaction mold was then assembled by use of the aligning studs, and the two halves were firmly bolted together. The lower piston was held in place with a steel pin.

As the details for the preparation of the test specimens varied in certain instances for the two factors of aggregate gradation and specimen height, various parts of the following procedure will be subdivided to account for these differences.

1. The pans of aggregate into which an armored thermometer had been inserted were placed in the heated oven (the oven regulator was set at 450 F).

Fig. 4 Cylindrical Mold and Typical Test Specimens; Triaxial, Modified Triaxial, and Stabilometer

a. 10-in. specimen - each batch pan contained only one-half the amount required for a test specimen.

b. 4-in. specimen - each batch pan contained the total amount needed to form a test specimen.

2. When the temperature of the aggregate was approximately

a. 225 F for the open-graded aggregate,

b. 125 F for the one-size aggregate,

the pan of asphalt cement containing another armored thermometer was placed in the oven. At the same time the assembled compaction mold, the upper compaction piston, the mixing bowl, the mixing paddle, the round steel bar, a flat shallow pan, and two large spoons were also placed in the hot oven.

3. All the materials and equipment were heated until the temperatures of the aggregate and the asphalt cement were

a. 300 ± 10 F and 275 ± 10 F, respectively, for the open-graded aggregate.

b. 200 ± 10 F and 235 ± 10 F, respectively, for the one-size aggregate.

An attempt was made to have the aggregates and the asphalt cement reach their required temperatures at the same time. Overheating of the asphalt cement was avoided as this would have changed its characteristics.

4. When the materials reached the proper temperature, the heated mixing bowl was placed on the beam balance which

was then tared. The aggregate from one batch pan was placed in the mixing bowl, and its weight to the nearest gram was determined. This provided a check on the aggregate batching operation as any discrepancy in the weight would indicate either the absence of various aggregate fractions or the improper weights of any aggregate sizes.

5. The weight of asphalt cement to be incorporated with the aggregate was added to the beam, and the hot material was poured into the metal mixing bowl until the beam was balanced.

6. For the 10-in. specimens, the pan of asphalt cement was returned to the oven to regain the designated temperature since the material for the specimen was mixed in two separate batches.

7. The mixing paddle was removed from the oven and placed in position on the Hobart mixer.

8. With the mixer operating at the slow speed, the asphalt cement and aggregate contents of the mixing bowl were mixed for two minutes as measured by a stop watch.

9. After the mixing operation was completed,

a. the hot bituminous-aggregate mixture for the 10-in. specimens was placed in a flat metal pan that had been heated in the oven. To prevent this half of the material needed for one test specimen from cooling during the mixing of the other portion, this pan containing the mixture was placed on the bottom shelf of the oven with the door remaining open.

was then tared. The aggregate from one batch pan was placed in the mixing bowl, and its weight to the nearest gram was determined. This provided a check on the aggregate batching operation as any discrepancy in the weight would indicate either the absence of various aggregate fractions or the improper weights of any aggregate sizes.

5. The weight of asphalt cement to be incorporated with the aggregate was added to the beam, and the hot material was poured into the metal mixing bowl until the beam was balanced.

6. For the 10-in. specimens, the pan of asphalt cement was returned to the oven to regain the designated temperature since the material for the specimen was mixed in two separate batches.

7. The mixing paddle was removed from the oven and placed in position on the Hobart mixer.

8. With the mixer operating at the slow speed, the asphalt cement and aggregate contents of the mixing bowl were mixed for two minutes as measured by a stop watch.

9. After the mixing operation was completed,

a. the hot bituminous-aggregate mixture for the 10-in. specimens was placed in a flat metal pan that had been heated in the oven. To prevent this half of the material needed for one test specimen from cooling during the mixing of the other portion, this pan containing the mixture was placed on the bottom shelf of the oven with the door remaining open.

b. the hot bituminous-aggregate mixture for the 4-in. specimens was transferred directly from the mixing bowl to the compaction mold by a heated spoon, following the method given in Part 12.

10. For the taller specimens requiring the mixing of the material in two separate batches, a new tare weight of the mixing bowl was determined on the beam balance since the bowl contained a small amount of the previous mixture. The mixing procedure was an exact duplication of that specified for the first half of the bituminous-aggregate mixture. After the mixing operations were completed, the second portion of the mixture was then placed in the flat pan with the first portion. These two portions were thoroughly blended by mixing with the heated spoon.

11. Near the end of the mixing operations, the heated compaction mold and the heated upper piston were removed from the oven and placed on the work bench. A 4-in. diameter circular disc cut from brown wrapping paper was inserted in the bottom of the mold to prevent the finished specimen from adhering to the lower piston.

12. The compaction mold was filled in

a. four equal layers for the 10-in. test specimens.

b. two equal layers for the 4-in. test specimens.

Each layer was rodded 25 times with a 3/4-in. round steel bar that weighed 1.4 lb. After the rodding of the last layer, another circular paper disc was placed in the mold to prevent

any bond between the specimen and the upper piston. This step was completed by inserting the upper piston into the compaction mold. In one phase of this study, one-size specimens with density values greater than that produced by the normal procedures were desired. To produce these, each layer, after rodding, was further compacted by placing the compaction foot of the pneumatic vibrator around the surface of the mixture layer for one minute, with the pneumatic vibrator operating at a pressure of 60 psi. This pneumatic vibrator with the compaction foot is illustrated in Fig. 5.

13. The filled compaction mold was centered in the hydraulic compaction device. This compaction device with the compaction mold in place is shown in Fig. 6. When the steel pin holding the lower piston in place was removed, both the upper and the lower pistons were free to move inside the compaction mold, thereby permitting the specimen to be compacted from both ends.

14. a. In the double-plunger compaction of the open-graded specimens, the hydraulic jack with the tonnage selector initially set at 15 tons was used to gradually apply pressure to the upper and lower pistons. When the pressure reached the 1500 psi mark on the gage, the tonnage selector was shifted to the 50-ton setting. The loading was continued until the gage pressure read 2500 psi, thus obtaining a pressure of approximately 2170 psi on each end of the specimen.

b. In the compaction of the one-size specimens by the double-plunger method, the hydraulic jack with the tonnage

Fig. 5 Pneumatic Vibrator with Compaction Foot and Specimen Mold

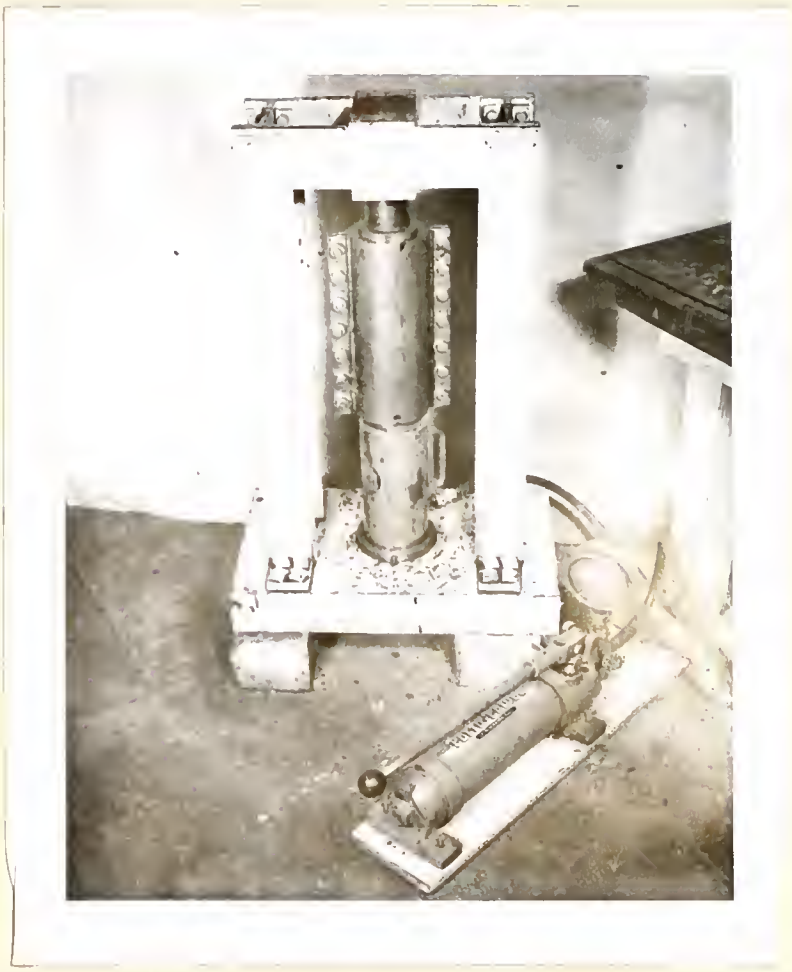


Fig. 6 Hydraulic Jack with Cylindrical Mold in Place for Double-Plunger Compaction

selector set at 15 tons was used to gradually apply a pressure to the upper and lower pistons until a value of 500 psi was obtained on the gage. This produced a pressure of approximately 430 psi on each end of the specimen. To provide better compaction, the compaction mold was vibrated by striking it with a leather mallet for 50 blows, while the compacting load of 500 psi was maintained.

15. These pressures were maintained constant for one minute (as measured by a stop watch) and then released.

16. The compaction mold containing the compacted specimen was immersed in a circulating cold-water bath for a period of 8 to 12 min.

17. After the cooling period, the compaction mold was replaced in the hydraulic compaction device, and a slight pressure was applied to the upper piston. This forced the specimen to move a short distance inside the mold, thus breaking the bond between the specimen and the sides of the mold.

18. After the compaction mold was removed from the hydraulic compaction device, the specimen was removed by disassembling the mold. The newly formed test specimens were placed in a convenient storage area at room temperature until the time for testing. However, the one-size specimens would not retain their original shape at room temperatures for more than four hours. Therefore, light-weight, sheet metal containers were placed around these specimens as soon as

they were taken from the compaction mold. These containers were not removed until the specimens were to be tested.

Typical specimens with both the open-graded and the one-size aggregates and a container for the one-size specimens are pictured in Fig. 7.

Density Determinations

In determining the densities and the change in volume of the triaxial test specimens, the following apparatus were used:

1. Torsion balance - capacity of 4.5 kg.
2. 15-gal can filled with water.
3. Suspension rod.
4. Suspension basket.
5. Polyethylene plastic sheeting - thickness of 0.002 in.
6. Paraffin.
7. Dial indicator - 0.001-in. scale divisions.
8. Support stand.

During the triaxial compression test of the open-graded specimens, there was no measurable evidence of any change in the volume of the sample. Therefore, it was assumed that the volume of the specimen remained constant throughout the triaxial test. Before the actual testing, each open-graded specimen was weighed to the nearest 0.1 g on the torsion balance. The height of the specimen was determined to the nearest 0.01 in. by averaging six measured heights obtained

Fig. 7 Open-Graded and One-Size Mixture Specimens and One-Size Specimen Container

from reading the referenced dial indicator mounted on the support stand, as shown in Fig. 8. These values thus permitted the density of the specimen to be computed.

However, the one-size specimens experienced a measurable reduction in volume when they were subjected to the triaxial compression test. Since a measure of this volume change was required for the proper reduction of the triaxial test data, the volumes (and also the densities) of the specimen both before and after the test were determined. The accepted procedure (47) of coating the porous specimen with melted paraffin during the density evaluation was not feasible as the paraffin flowed into the voids of the mixture and produced erroneous results. Therefore, the following procedures were adopted to measure the densities and the volumes of the one-size specimens with an acceptable degree of accuracy.

1. The weight in air of the test specimen was determined on the torsion balance. (All weight readings were obtained to the nearest 0.1 g.)

2. The height of the specimen was determined to the nearest 0.01 in. by averaging six measured heights obtained from reading the referenced dial indicator mounted on the support stand, as shown in Fig. 8.

3. The cylindrical surface of the test specimen was then tightly wrapped in a thin sheet of polyethylene plastic, and the weight in air of the specimen plus the plastic sheeting was recorded.

Fig. 8 Specimen Height Determination

4. After the ends of the specimen and the plastic seams were sealed by brushing with melted paraffin so as to completely water-proof the specimen, the weight in air of the encased specimen was determined.

5. After taring the torsion balance for the suspension rod and the suspension basket which was completely submerged in the water contained in the 15-gal can, the encased specimen was placed in the suspension basket, and the weight of the specimen in water at room temperature was obtained. The laboratory setup for the density determination is pictured in Fig. 9.

6. After determining these four weights both before and after the triaxial compression test and knowing the specific gravity values of the polyethylene plastic and the paraffin, the volumes and the densities of the specimen before and after the triaxial test and the change in volume were calculated as shown by the sample data sheet in Fig. 10.

7. The specimen was prepared for the triaxial compression test by removing the casing of polyethylene plastic and paraffin. Those that had been tested were discarded without salvaging the plastic sheeting and the paraffin.

Triaxial Compression Test

In performing the triaxial compression test on the laboratory compacted test specimens the following pieces of apparatus were utilized:

Fig. 9 Equipment for Specimen Density Determination

DATA SHEET

Density Determinations - Bituminous Mixtures

Specimen No. S-25-B	Before Test		After Test
Height	3.96	in.	3.18
Wt of Specimen in Air	1421.8	g	1420.3
Wt of Specimen & Plastic in Air	1424.4	g	1423.2
Wt of Specimen, Plastic, & Paraffin in Air	1456.6	g	1452.0
Wt of Encased Specimen in Water	611.5	g	646.3
Vol of Encased Specimen (3 - 4)	845.1	cc	805.7
Sp gr of Polyethylene Plastic	0.92		
Sp gr of Paraffin	0.88		
Vol of Plastic (2 - 1)/6	2.8	cc	3.2
Vol of Paraffin (3 - 2)/7	36.6	cc	32.7
Vol of Specimen (5 - 8 - 9)	805.7	cc	769.8
Density of Specimen 1/10 x 62.4	110.1	lb/cu ft	115.1

Fig. 10

1. Riehle mechanical testing machine with a variable speed drive - capacity of 50,000 lb.
2. Proving rings - capacities of 4000 lb and 10,000 lb.
3. Triaxial test cell.
4. Rubber membranes - diameter of 4 in., length of 12 in., and thickness of 0.015 in.
5. Cenco vacuum pump.
6. Bottled nitrogen.
7. Airco nitrogen regulator.
8. Polyethylene plastic sheeting - thickness of 0.010 in.
9. Dial indicator - 0.001-in. scale divisions.
10. Support bracket.
11. Copper tubing.
12. Vacuum hose.
13. Rubber bands.
14. Room thermometer.

General views of the complete assembly for the triaxial compression test are shown in Fig. 11 for the rational test and in Fig. 12 for the irrational or modified test. A schematic diagram of the triaxial test cell is illustrated in Fig. 13. The compression load, as measured by a proving ring, was transferred to the test specimen through the vertical loading piston and the upper loading plate. A ball bushing assembly with a grease seal in the cover plate permitted the load to be applied with both a negligible amount of side friction and a negligible loss of nitrogen from the triaxial cell. As a safety measure, a relief valve set at

Fig. 11 Rational Triaxial Compression Test

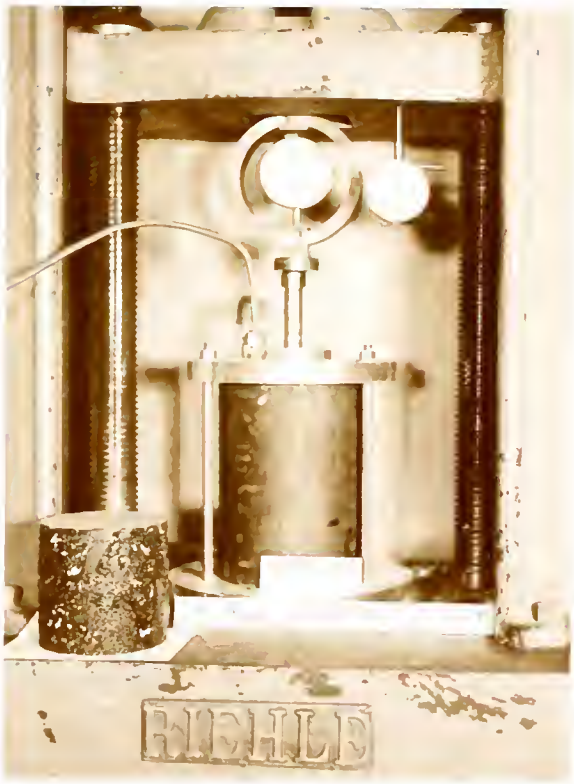


Fig. 12 Irrational Triaxial Compression Test

DETAILS OF TRIAXIAL CELL

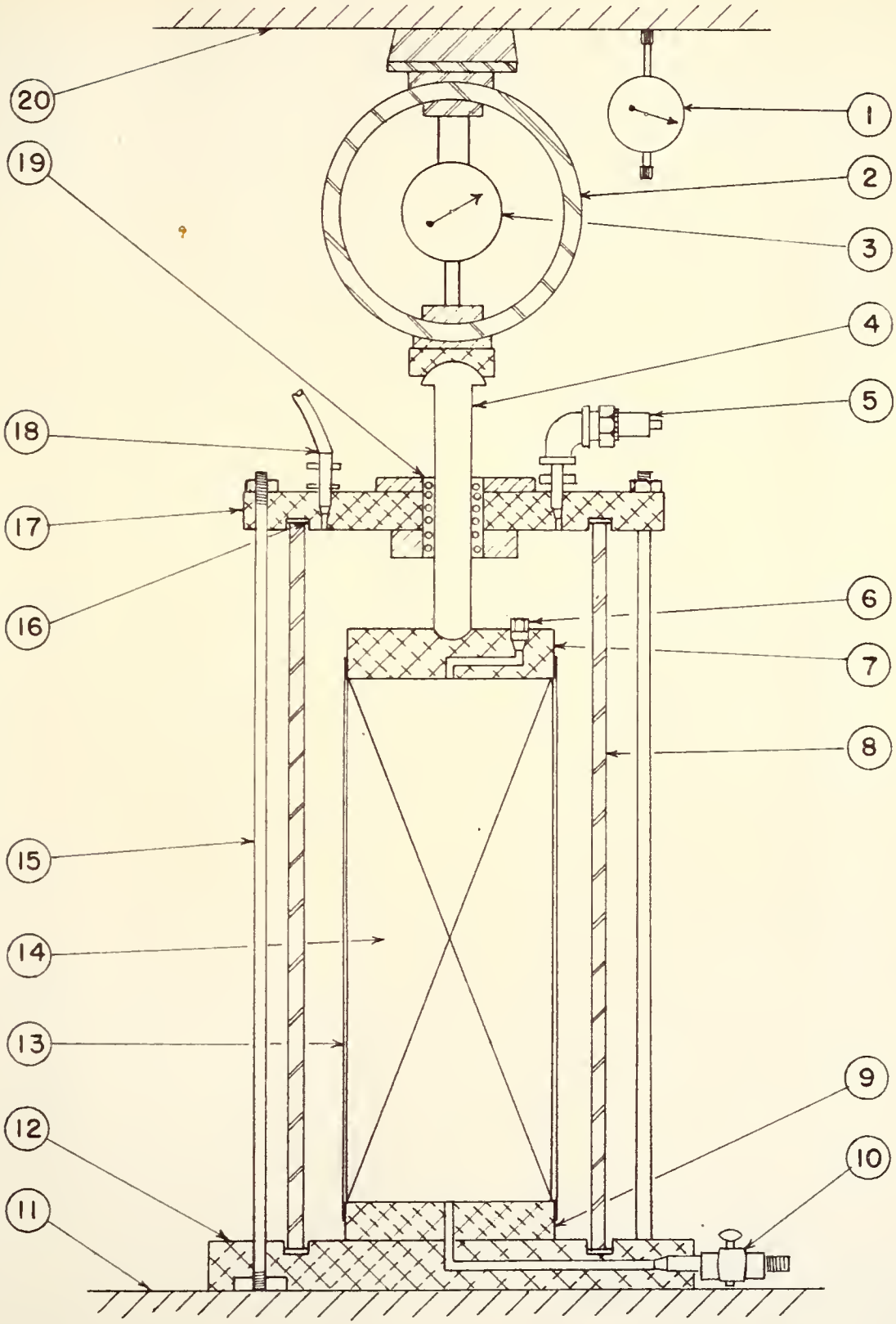


FIG. 13

Key to Figure 13

1. Vertical-movement dial indicator.
2. Proving ring.
3. Load dial indicator.
4. Loading piston.
5. Relief valve.
6. Upper drainage connection.
7. Upper loading plate.
8. Steel cylindrical wall of the triaxial cell.
9. Lower loading plate.
10. Lower drainage connection.
11. Lower platen of the testing machine.
12. Base of the triaxial cell.
13. Rubber membrane.
14. Compacted bituminous-aggregate test specimen.
15. Threaded connecting rod.
16. Rubber gasket.
17. Cover plate.
18. Compressed nitrogen inlet.
19. Ball bushing with grease seal.
20. Upper platen of the testing machine.

195 psi was installed in the cover plate. Because the dial indicator was referenced to the upper platen of the testing machine, the corresponding load dial indicator reading was subtracted from the deflection dial reading to obtain a true measure of the vertical movement of the test specimen. Since only open-graded and one-size bituminous-aggregate specimens were tested in the triaxial apparatus, just the lower drainage system was required to evacuate the specimen for the leakage check before the test and to indicate any leakage and pore pressure build-up during the test. Therefore, the upper drainage connection was sealed off.

At certain times the variable of testing temperature was difficult to maintain within the prescribed range of 77 ± 5 F since no temperature control devices were available. After the initial density determination had been completed and with the specimen at the correct room temperature (in the range of 77 ± 5 F), the triaxial compression test was performed on the laboratory compacted specimen in the following detailed manner:

1. A rubber membrane obtained from Soiltest, Inc., was rolled up and fitted over the lower loading plate.

a. If the confining pressure was 60 psi or greater, the open-graded specimens were wrapped in a sheet of polyethylene plastic to prevent the rupturing of the rubber membrane.

b. If the confining pressure was 30 psi or greater, the one-size specimens were wrapped in a sheet of polyethylene

plastic to protect the rubber membrane against rupture. A series of check tests at a confining pressure of 90 psi showed that the use of the polyethylene plastic wrapping produced no measurable influence on the results of the triaxial compression tests performed on open-graded rational specimens as compared to identical specimens tested under identical conditions without the provision of a plastic wrapping.

2. After the upper loading plate was placed on top of the specimen, the rubber membrane was rolled up over the specimen and this loading plate. Several rubber bands secured the ends of the rubber membrane to the upper and the lower loading plates.

3. A vacuum hose was attached to the lower drainage connection which was open. The air inside the test specimen was slowly evacuated with the Cenco vacuum pump until the rubber membrane was tight around the specimen. Then the lower drainage line was closed, and the vacuum hose was removed. This operation and the position of the specimen in the triaxial test cell are illustrated in Fig. 14.

4. The rubber membrane was assumed to be air-tight if the vacuum was maintained for a period of at least five minutes. However, if the vacuum did not remain within this interval, an air leak was indicated. The source of this leak was located, and the leakage was eliminated before the actual testing operations commenced.

Fig. 14 Preparation for the Triaxial Compression Test

5. After an air-tight rubber membrane was secured, the triaxial test cell was completely assembled by placing the steel jacket around the specimen and by connecting the cover plate to the base with three threaded connecting rods. The assembly was so aligned that the lower hemispherical end of the loading piston was seated in the hemispherical recession in the upper loading plate. The radius of the recession in the plate was much larger than that for the lower end of the piston.

6. The triaxial test cell was centered in the Riehle testing machine, and the upper platen was lowered to bring the proving ring in contact with the upper end of the loading piston. Likewise, the hemispherical end of the piston had a radius smaller than that of the hemispherical recession at the bottom of the proving ring. The copper tubing was then attached to connect the triaxial test cell with the regulated supply of bottled nitrogen.

7. With the lower drainage connection open to the atmosphere, the desired confining pressure was applied to the test specimen by setting the Airco nitrogen regulator at the correct mark.

8. The deflection dial was held in position by a support bracket attached to the testing machine frame. This dial was placed so that there was initially a positive contact with the upper platen of the testing machine.

9. After setting the load dial and the deflection dial to read zero, the test specimen was loaded at the rate of

0.05 in. per min. As the test progressed, simultaneous readings of both the load and the deflection dials were obtained periodically. A sample data sheet is presented in Fig. 15.

10. The loading was continued until the desired amount of strain was reached. At the end of the test the testing machine was stopped, the nitrogen was released from the triaxial cell, and the load was removed by raising the upper platen of the testing machine.

11. The triaxial test cell was then disassembled, and the test specimen was removed.

Data Reduction

After the triaxial compression test was completed, a graph of the load dial reading as the ordinate versus the vertical deflection dial reading as the abscissa was plotted. By drawing the best straight line through the linear portion of these data, a correction was obtained so that the deflection would be zero for the no-load condition. By applying this correction and by subtracting the load dial reading from the deflection dial reading, the true vertical deflection of the test specimen was determined.

The following portion of the reduction of the laboratory test data will be discussed separately for the open-graded and the one-size test specimens as the procedures differed for these two bituminous-aggregate mixtures.

DATA SHEET

Triaxial Test - Bituminous Mixtures

Sample No. S-25-B

Date Tested 12/27/56 By J.O.& M.S.

Initial Ht 4.00 in.

Rate of Load 0.05 in./min

Weight 1428.0 gm

Temperature 78 F

Initial Dia 4.00 in.

Confining Pressure 90 psi

Unit Wt 108.2 lb/cuft

Proving Ring No. 10,000

Load Dial	Strain Dial	Load	Corr. Strain	% Strain	Corr. Area	Dev. Stress	Total Stress
0	0	0	0	0	12.57	0	
10	28	150	4	0.10	12.57	12	102
20	34	250	9	0.22	12.57	20	110
40	46	350	19	0.48	12.60	28	118
60	64	950	35	0.88	12.66	75	165
73	80	1200	60	1.50	12.73	94	184
85	100		69	1.72	12.76		
94	120	1750	88	2.20	12.80	137	227
101	140		106	2.65	12.85		
107	160	1950	126	3.15	12.90	151	241
113	180		146	3.65	12.95		
120	200	2200	165	4.12	13.01	169	259
125	220		185	4.62	13.07		
130	240						
134	260	2475	224	5.60	13.18	188	278
137	280						
141	300	2600	263	6.58	13.29	196	286
149	350						
159	400	3000	361	9.02	13.59	221	311
173	450						
182	500	3450	459	11.48	13.91	248	338
191	550						
201	600	3850	557	13.92	14.23	271	361
210	650						
219	700	4250	655	16.38	14.56	292	382
229	750						
238	800	4650	753	18.82	14.93	311	401
243	850						
251	900	4900	852	21.30	15.33	320	410

Fig. 15

In the triaxial stability analysis of the open-graded test specimens, the volume of the specimen was assumed to remain constant as there was no measurable evidence of any change in the volume during the triaxial test. Upon this assumption, the expression for the corrected cross-sectional area of the specimen experiencing no volume change is derived in Appendix A. Since this corrected area was a function of the initial specimen height, the average heights of the rational and the irrational test specimens were calculated. For these average values the curves in Fig. 32 and Fig. 33 were plotted to show the corrected cross-sectional area of the specimen as a function of the change in the specimen height.

During the triaxial compression tests on the one-size laboratory compacted specimens, the volume of the specimen was reduced a measurable amount, thus increasing the density of the specimen as the test progressed. The expression for the corrected cross-sectional area of the specimen experiencing a volume reduction is derived in Appendix B upon the assumption that the reduction in volume is directly proportional to the change in the specimen height. Like the open-graded specimens, the average heights of the various groupings of the test specimens were computed since the corrected area was dependent upon the initial specimen height. By dividing the total change in the specimen volume in cubic inches by the total change in the specimen height in inches, the constant "C" in square inches was evaluated for each

test specimen. As this relative measure of the degree of volume change varied over a limited range for the individual specimens of a given grouping, a parameter of curves depending on different "C" values was plotted for each of the average initial-specimen-height groups. These curves give the corrected cross-sectional area of the specimen as a function of the change in the specimen height, and are presented in Fig. 34 to Fig. 39.

The remaining portion of the data reduction is similar for the test specimens having either the open-graded aggregate or the one-size aggregate. Having determined the corrected cross-sectional area, the corresponding load in pounds was divided by this area in square inches in order to calculate the deviator stress in pounds per square inch. The total normal stress in pounds per square inch was computed by adding the confining pressure in pounds per square inch to the deviator stress. The strain for each reading was found by dividing the true vertical deflection by the initial height of the specimen. A stress-strain diagram was prepared for each triaxial test; an example of this diagram is shown in Fig. 16. Obtained from the stress-strain curves, the total normal stresses for various percent strains and the peak condition (if it existed) were tabulated as the results under the various test-variable groupings.

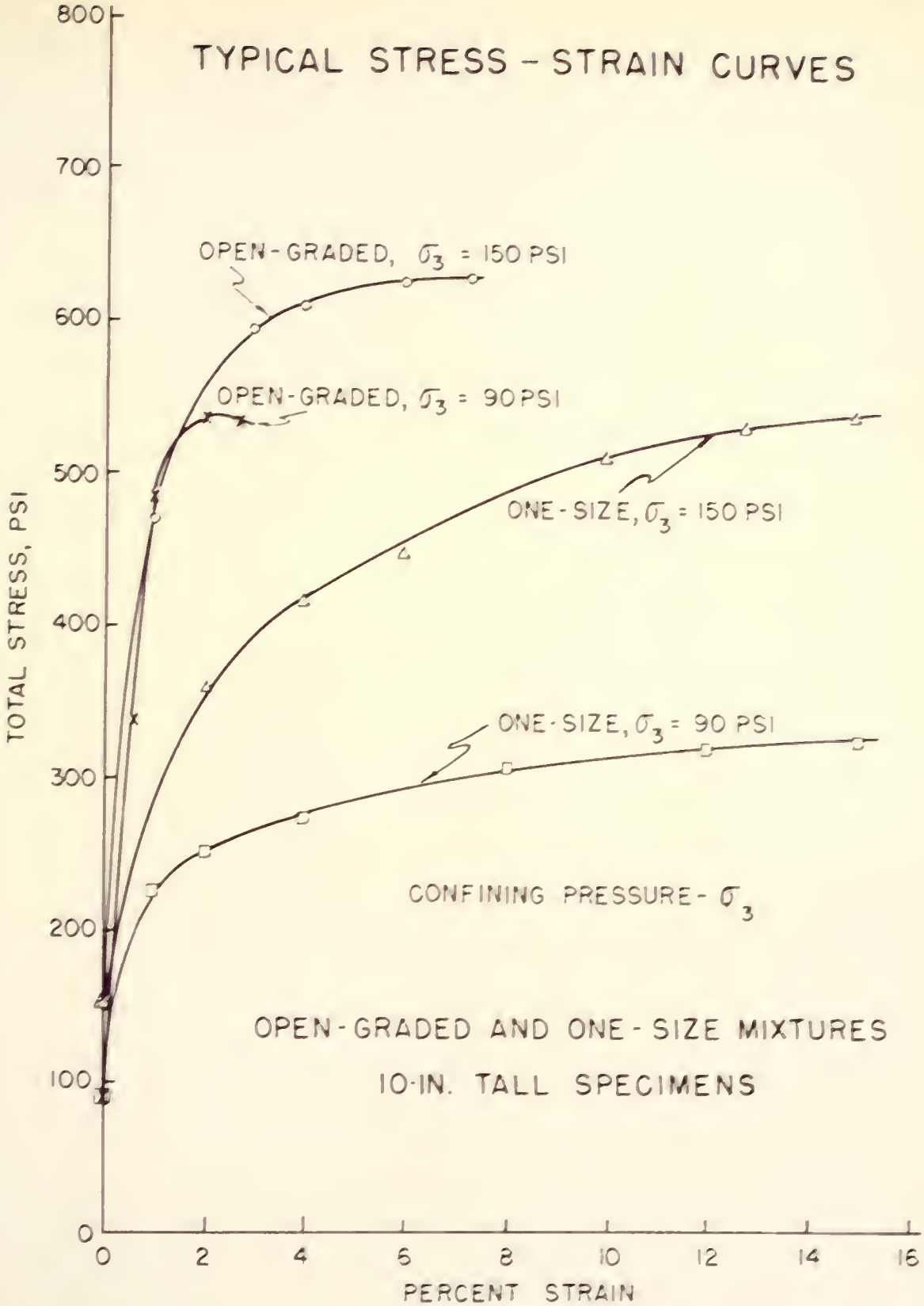

TYPICAL STRESS - STRAIN CURVES
OPEN-GRADED, σ_3 = 150 PSI
OPEN-GRADED, σ_3 = 90 PSI
ONE-SIZE, σ_3 = 150 PSI
ONE-SIZE, σ_3 = 90 PSI
CONFINING PRESSURE - σ_3
OPEN-GRADED AND ONE-SIZE MIXTURES
10-IN. TALL SPECIMENS
TOTAL STRESS, PSI
PERCENT STRAIN
0
100
200
300
400
500
600
700
800
0
2
4
6
8
10
12
14
16

FIG. 16

RESULTS

In this laboratory investigation, triaxial stability values were determined for compacted bituminous mixtures by the open-system triaxial compression test using confining pressures up to 150 psi. The variable conditions introduced were aggregate gradation, confining pressure, specimen height, degree of compaction, type of aggregate, and asphalt content. The results of the triaxial compression tests performed on the bituminous-aggregate mixtures for the various testing variables are presented in Appendix C, Tables 6 to 15.

In addition, Stabilometer stability values were obtained for the variable conditions of aggregate gradation, type of aggregate, and asphalt content. These three variables were identical for the triaxial and the Stabilometer tests. This control permitted the results from the two testing procedures to be compared directly. The results of the Stabilometer tests are given in Appendix D, Tables 16 to 19.

The two aggregate gradations, open-graded and one-size, the type of test, and the variables studied for each test are outlined as follows:

I. Open-graded.

A. Triaxial compression test.

1. Crushed limestone.

a. Confining pressure.

b. Specimen height.

(1) Rational.

(2) Irrational.

c. Asphalt content.

2. Uncrushed gravel and natural sand.

a. Confining pressure.

b. Asphalt content.

B. Stabilometer test.

1. Crushed limestone - asphalt content.

2. Uncrushed gravel and natural sand - asphalt content.

II. One-Size.

A. Triaxial compression test.

1. Crushed limestone.

a. Confining pressure.

b. Degree of compaction.

(1) Double-plunger method.

(2) Double-plunger with vibration method.

c. Specimen height.

(1) Rational.

(2) Irrational.

d. Asphalt content.

2. Uncrushed gravel and natural sand.

a. Confining pressure.

b. Asphalt content.

B. Stabilometer test.

1. Crushed limestone - asphalt content.

2. Uncrushed gravel and natural sand - asphalt content.

The results, presented in both tabular and graphical forms, and their significance are discussed in the remaining portion of this section according to the above outline.

The selection of the two main divisions, open-graded mixture and one-size mixture, for this research report was predicated on the findings that the stress-strain curves representing these two bituminous mixtures were considerably different. As shown in Fig. 16, the stress-strain diagrams for the open-graded mixture reached a peak normal stress value at a relatively low strain value, while the normal stress continued to increase at a decreasing rate with an increase in the strain value for the one-size mixture.

Open-Graded Mixture

Rational Triaxial Test. The shearing resistance of the open-graded paving mixture is illustrated in Fig. 17 where the confining pressure versus the peak normal stress was plotted from stress values ascertained from rational triaxial tests with confining pressures ranging from 0 to 150 psi, at 30 psi intervals. Crushed limestone was used in this mixture at an asphalt content of 5.0 percent by weight of the total mixture. This bituminous-aggregate mixture possessed appreciable triaxial stability, 390 psi, at the low confining pressure of 30 psi. The shearing strength increased linearly with confining pressure until a value in excess of 700 psi was reached at the 150-psi level. Since this linear

CONFINING PRESSURE VS. PEAK NORMAL STRESS

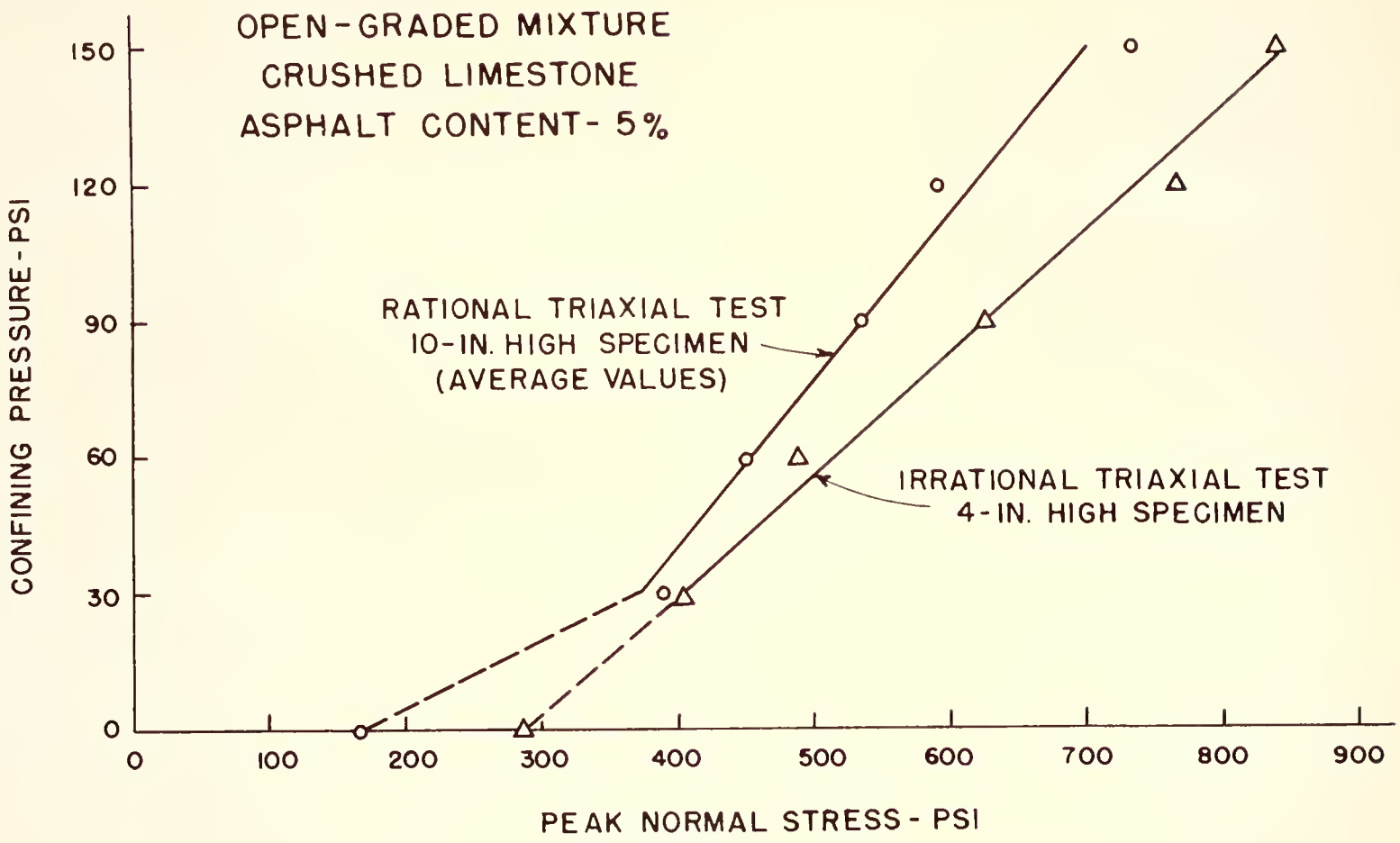


FIG. 17

relationship between confining pressure and peak normal stress is analogous to a straight-line Mohr rupture envelope (1), the shearing strength of this bituminous mixture may be ascertained by the Coulomb equation and represented by the constant fundamental strength properties, (c) cohesion and (ϕ) angle of internal friction. However, the peak normal stress at 0 psi confining pressure did not agree with the linear relationship, thus indicating that the Mohr rupture envelope is curvilinear in the range of very low confining pressures (less than 30 psi). This condition has been validated by previous rational triaxial test investigations in both the soil mechanics and the bituminous-mixture areas.

Specimen Height (Irrational Triaxial Test). The results obtained in evaluating the effects of specimen height on the triaxial stability of bituminous paving mixtures tested at high confining pressures are also presented in Fig. 17. The relationship between the plotted stress values was best represented by a straight line. These irrational triaxial tests were performed on compacted bituminous specimens having the same type of aggregate and the same asphalt content as the specimens used in the previously discussed rational triaxial test, except that 4-in. high specimens were used (height to diameter ratio was less than two). For corresponding confining pressures, the peak normal stresses obtained in the irrational triaxial tests were greater than those stresses determined in the rational triaxial testing.

This result was produced by the additional lateral support afforded the irrational test specimen by the shearing stresses developed between the ends of the specimen and the upper and the lower loading plates. This reasoning also explains the close position of the peak normal stress for the 0 psi confining pressure to the extension of the straight line representing the irrational triaxial tests in Fig. 17.

For both the rational and the irrational triaxial tests of the open-graded mixtures, the peak normal stresses were reached at strain values of five percent or less. The strains at which the peak stability values occurred centered around one percent for the 30 psi confining pressure, and they gradually increased with an increase in the confining pressure to the 150-psi level where the strain values were near five percent (see Fig. 16).

Although the two series of test values plotted in Fig. 17 appear to be characterized by a linear relationship, the slopes of these two straight lines are different. This indicates that the rational and the irrational triaxial tests do not produce equal measures of the fundamental strength properties, cohesion and angle of internal friction, of this open-graded mixture.

Type of Aggregate. To evaluate the variable of aggregate type for rational triaxial testing at high confining pressures, 10-in. high specimens of the open-graded mixture were formed from uncrushed gravel and natural sand at asphalt contents of 4.0 and 5.0 percent by weight of the total mixture.

A comparison of the triaxial strengths of these specimens with those values obtained from crushed limestone specimens having identical asphalt contents is presented in Fig. 18 for the confining pressures of 30, 90, and 150 psi. The greater shearing strength exhibited by the crushed limestone mixture for the range of confining pressures was attributed to the larger degree of interlocking among these angular aggregate particles as compared to the uncrushed material.

Asphalt Content. The effects of high confining pressures on the triaxial stability were determined for the open-graded mixture at asphalt contents of 4.0, 5.0, and 6.0 percent by weight of the total mixture. The results of the rational triaxial tests for the low and high confining pressures of 30 and 150 psi, respectively, are shown in Fig. 19 for the crushed limestone specimens. The analogous case for the uncrushed gravel and natural sand specimens is presented in Fig. 20. For both types of aggregate, it is readily apparent from these two figures that confining pressure value has little or no effect on the determination of the asphalt content producing maximum stability in the triaxial compression test. The asphalt content producing maximum triaxial stability appeared to be near 5.0 percent by weight of the total mixture for both the crushed limestone and the uncrushed gravel and natural sand mixtures, whether evaluated at 30 or 150 psi confining pressure.

Stabilometer Test. The Stabilometer test investigation of the open-graded mixture made from the two aggregate types

CONFINING PRESSURE VS. PEAK NORMAL STRESS

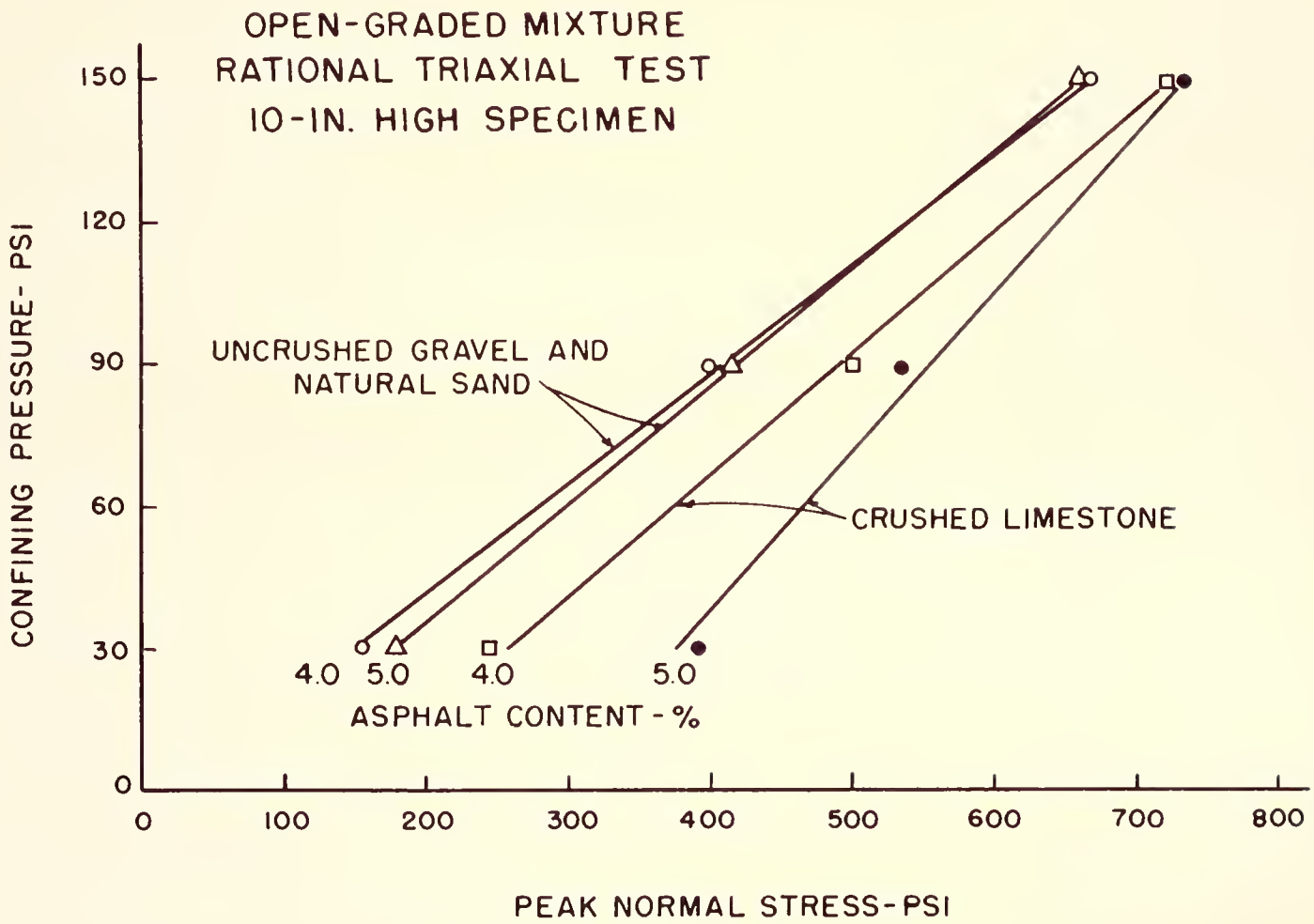


FIG. 18

NORMAL STRESS VS. ASPHALT CONTENT

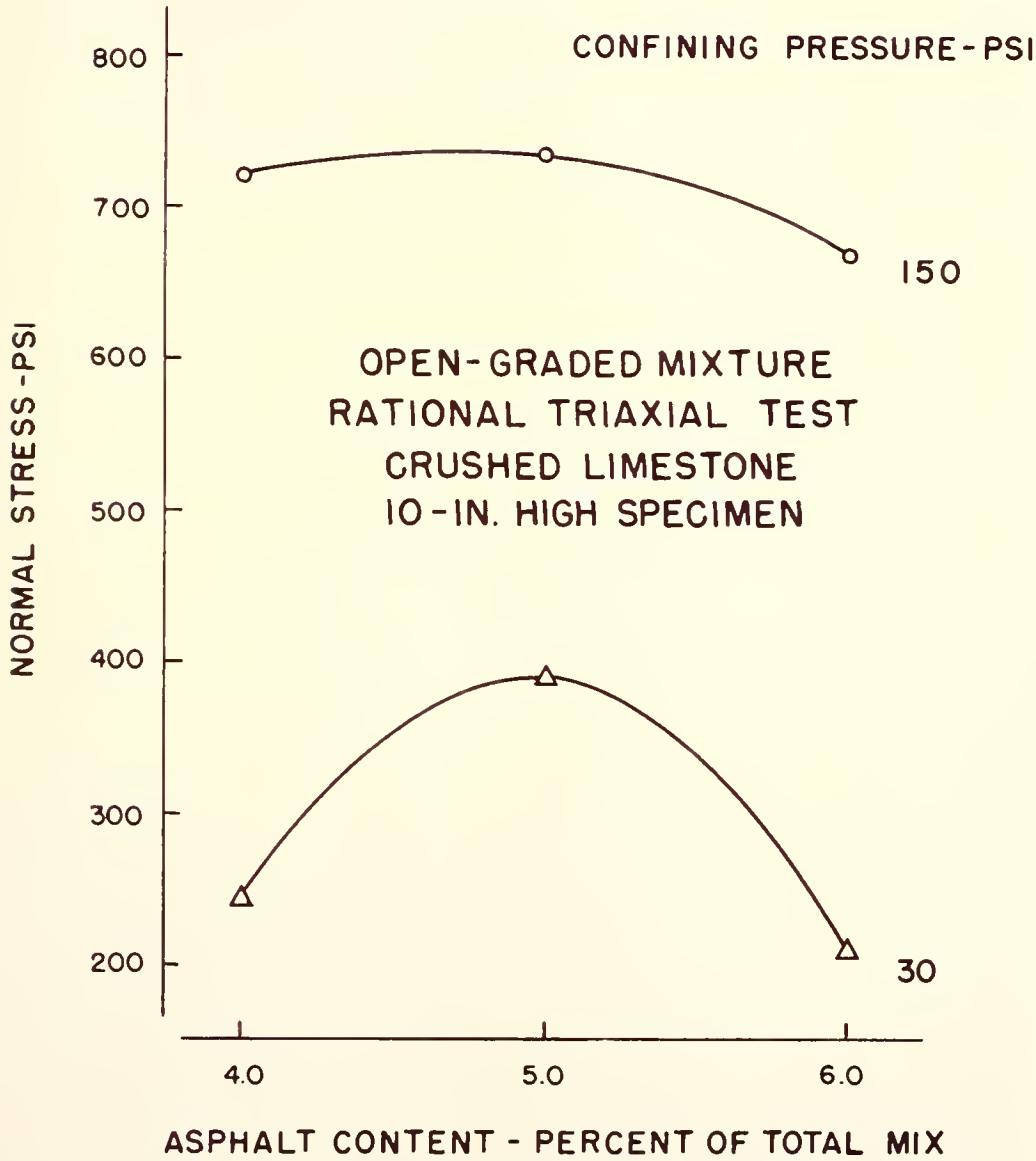


FIG. 19

NORMAL STRESS VS. ASPHALT CONTENT

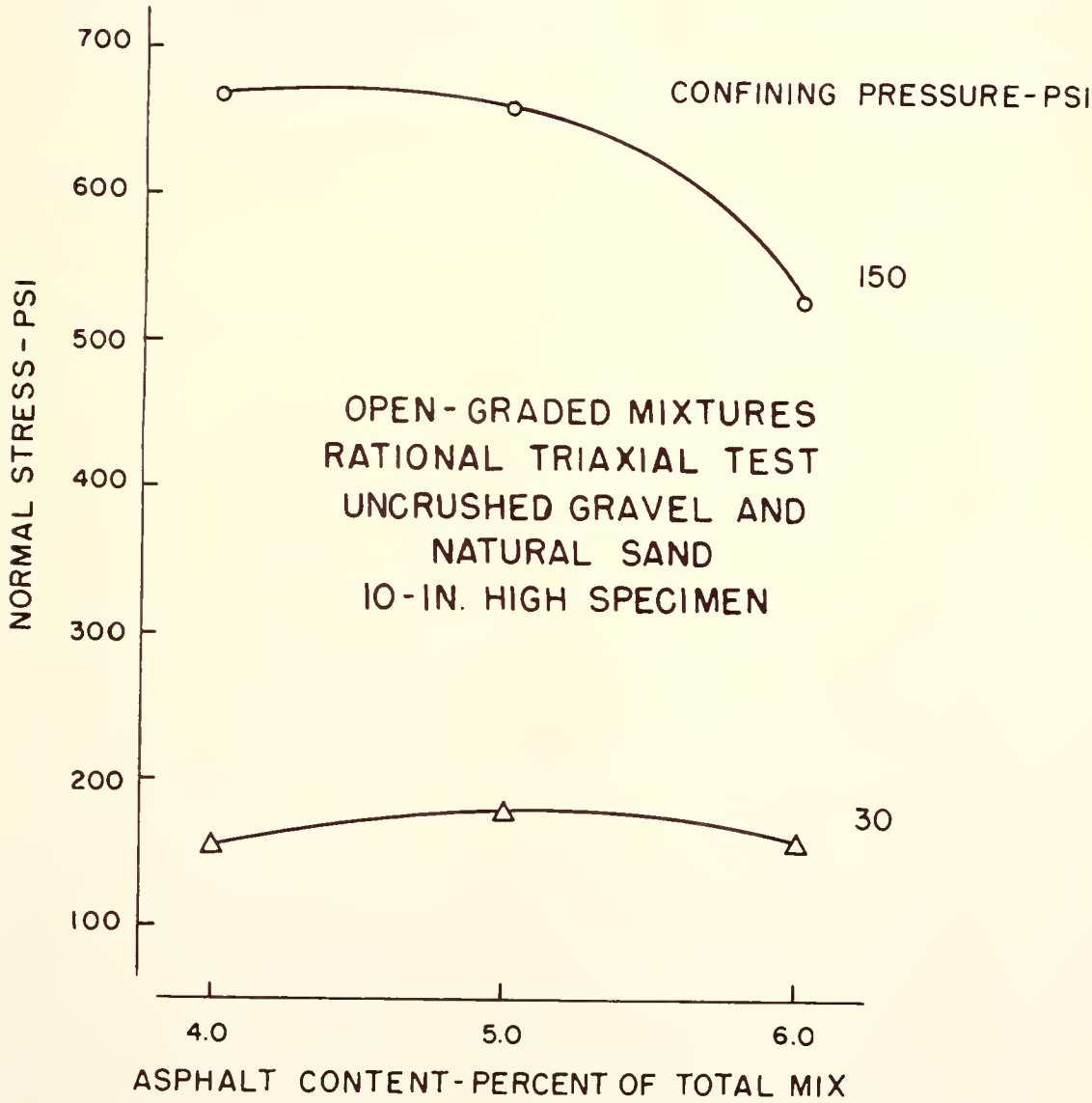


FIG. 20

for the same range of asphalt content as used for the triaxial testing revealed analogies similar to those deduced in the rational triaxial test. The Stabilometer results indicated that the stability of the crushed limestone mixture was greater than that of the uncrushed gravel and natural sand mixture, and that the optimum asphalt content was near 5.0 percent by weight of the total mixture for both types of aggregate. These results are illustrated in Fig. 21.

One-Size Mixture

Rational Triaxial Test. The other aggregate gradation investigated in this study of triaxial compression testing at high confining pressures was the one-size mixture. This paving mixture has performed more satisfactorily under the detrimental actions of traffic than that predicted performance indicated by laboratory appraisals for this bituminous-aggregate combination. The development of the shearing strength of this one-size mixture is illustrated in Fig. 22 where the confining pressure versus the normal stress at various percent strain was plotted. These stress values were determined by rational triaxial tests with confining pressures ranging from 0 to 150 psi, at 30 psi intervals, on compacted crushed-limestone specimens having an asphalt content of 5.0 percent by weight of the total mixture. As evidenced by the typical stress-strain curves for this one-size mixture at various confining pressures (see Fig. 16),

STABILOMETER STABILITY VS. ASPHALT CONTENT

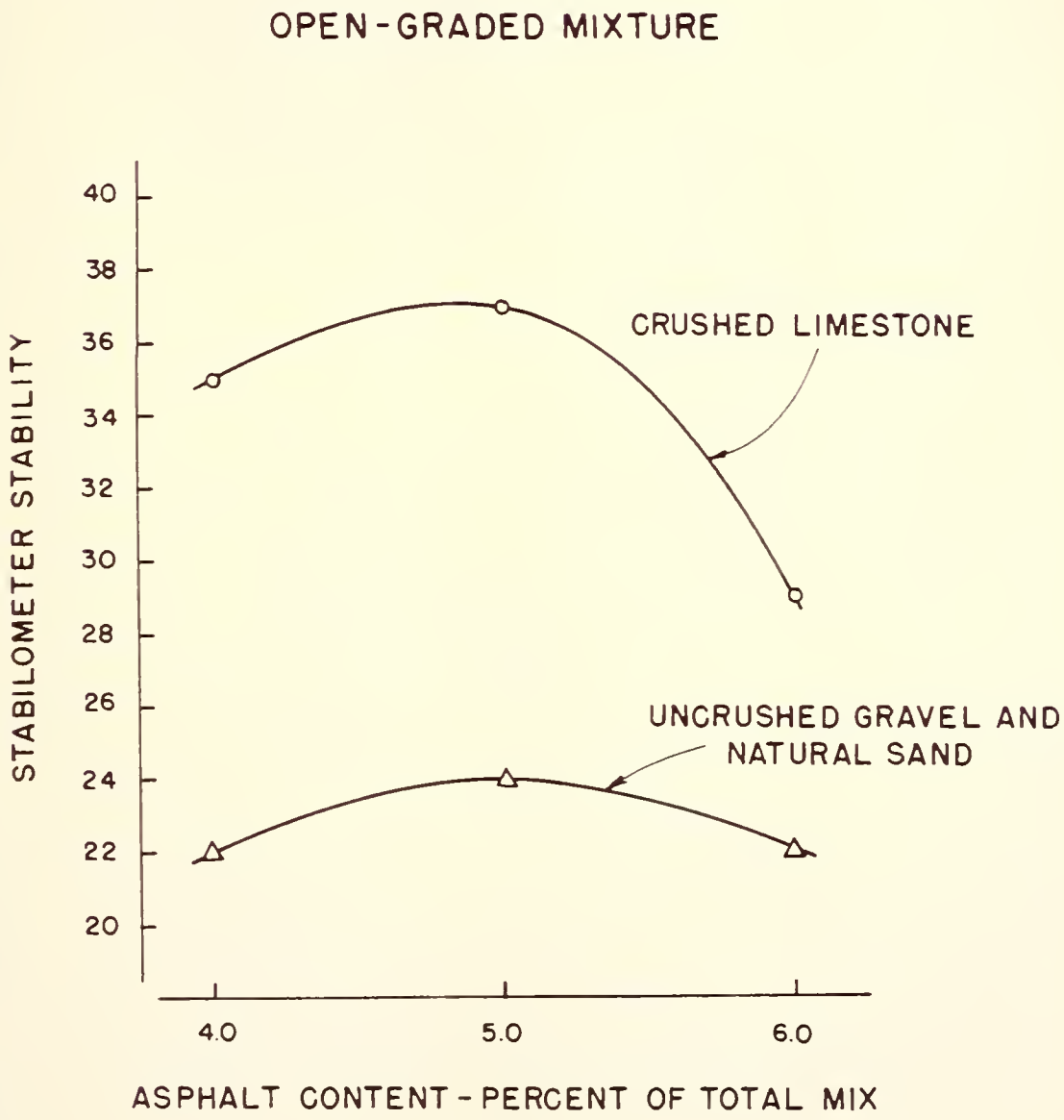


FIG. 21

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

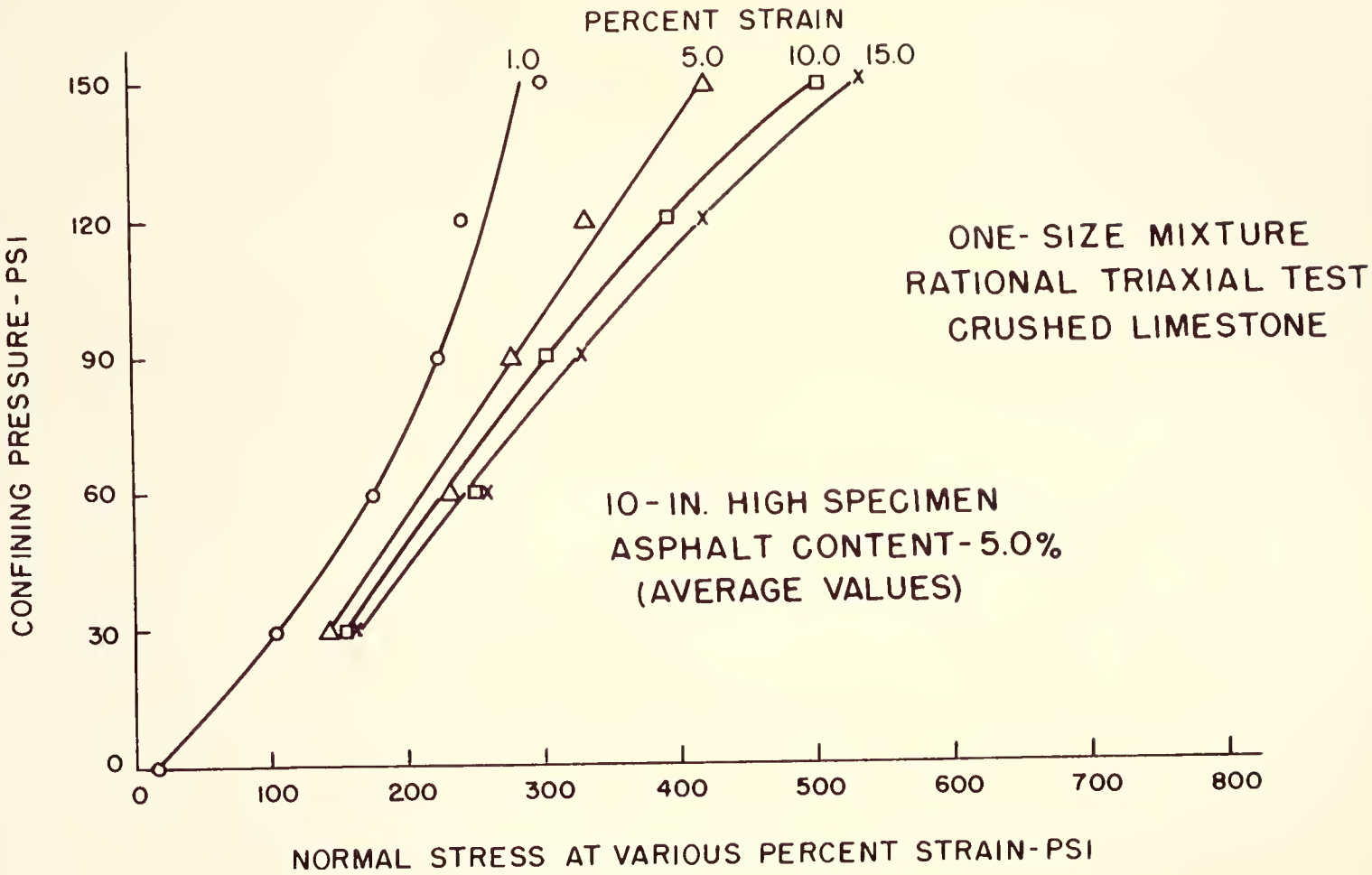


FIG. 22

the normal stresses increased at a decreasing rate with an increase in the amount of strain applied to the test specimen. However, this decreasing rate was found to vary inversely with the confining pressure. This is illustrated by comparing the normal stresses at 1.0 and 15.0 percent strain in Fig. 22 for the two confining pressures of 30 and 150 psi. Thus, at the 30-psi level shearing strengths of approximately 100 and 160 psi were developed at 1.0 and 15.0 percent strain, respectively; while the corresponding stability values at a confining pressure of 150 psi were 300 and 540 psi. The absence of any peak normal stress for positive confining pressures provoked the analyses of the triaxial strength data at 1.0, 5.0, 10.0, and 15.0 percent strain. The physical limitations of the triaxial test apparatus and the difficulty of forming completely homogeneous test specimens of the one-size mixture prevented the determination of normal stress at greater strain values. The value of 15.0 percent strain may also be considered the maximum practical deformation limit for flexible pavement design applications.

The relationship between the confining pressure and normal stress at 1.0, 5.0, 10.0, and 15.0 percent strain for the one-size mixture (see Fig. 22) is considerably different than that presented for the open-graded mixture (see Fig. 17). The normal stresses at 1.0 percent strain increase at a decreasing rate with an increase in the confining pressure. At the 5.0 percent strain value, the graph of these two stress

values is best represented by a straight line. The normal stresses at the larger strains, 10.0 and 15.0 percent, follow a curvilinear pattern in which the normal stresses increase at an increasing rate for an increase in the confining pressure. The variable nature of these stress diagrams indicates that the cohesion and the angle of internal friction were not constants in this one-size mixture when the stability was ascertained by the rational triaxial compression test. The angle of internal friction and/or the cohesion varied throughout the range of confining pressures and the range of strains. Thus, the Coulomb equation and fixed values of cohesion and angle of internal friction cannot truly represent the triaxial stability of this one-size mixture. Since the triaxial compression test on the one-size limestone mixture produced a peak normal stress near 1.0 percent strain for the 0 psi confining pressure condition, the other three sets of curves must show a reversal in slope in the range of low confining pressures and approach zero at 0 psi confining pressure.

Density Determination. During all the triaxial compression tests (except at 0 psi confining pressure) of the one-size mixture, a measurable reduction in the volume of the specimen occurred. To properly evaluate the triaxial test data, this rearrangement of the aggregate particles and the reduction in the volume were considered for the one-size specimens by determining their bulk densities both before and after the triaxial compression test.

The bulk density of a compacted-bituminous-mixture specimen is determined by computing the ratio of its weight in air to its bulk volume. The Asphalt Institute (47) has summarized the determination of the bulk volume of the specimen by one of the following procedures:

1. The bulk volume of a specimen having a dense, impermeable surface texture is determined as the difference between its weight in air and its weight in water.

2. The bulk volume of a specimen having an open, permeable surface texture is determined as the difference between its weight in air and its weight in water when coated with paraffin.

3. The bulk volume of a specimen having a dense, smooth surface texture and formed to dimensions that can be accurately measured is computed from the diameter and the height measurements of the specimen.

The last procedure was valid for the determination of the bulk density of the open-graded specimens. However, none of the three available methods were applicable to evaluate the changes in volume and density that the one-size specimens experienced when they were subjected to a triaxial compression test. As the compacted bituminous-aggregate specimen was changed from a uniform cylindrical shape to a very distorted shape as a result of the triaxial test, the method utilizing the measured dimensions was of no practical value. The very porous texture of the one-size mixture necessitated that the specimen be coated in some manner, thus eliminating the first procedure.

The method of coating the compacted specimen with paraffin was attempted, but this proved unsatisfactory as the melted paraffin flowed into the void space. This condition produced erroneous results as the density values were increased because the voids were decreased. Therefore, it was mandatory that a new method of density determination be developed. This procedure entailed the wrapping of the specimens in a carefully-cut thin sheet of polyethylene plastic and then sealing the seam and the ends of the specimen with paraffin. A detailed description of this method is presented under the heading, Density Determinations, in the section, Apparatus and Procedure.

Design Criterion. It logically may be reasoned that the one-size mixture develops increasing shearing strength with increases in the confining pressure and with increases in the degree of specimen deformation. In order to establish a logical failure criterion for this paving mixture, the rate at which the shearing resistance was mobilized was investigated. This shearing stress development is pictured in Fig. 23. The curve for each confining pressure appears to asymptotically approach a maximum normal stress. Since the normal stresses at 15.0 percent strain are reasonably close in each case to these maximum values, it was arbitrarily assumed that the normal stresses at 15.0 percent strain represented the condition of maximum shearing resistance. The ratios of the normal stresses at 1.0, 5.0, and 10.0 percent strain to the normal stresses at 15.0 percent strain for the range of

NORMAL STRESS VS. PERCENT STRAIN

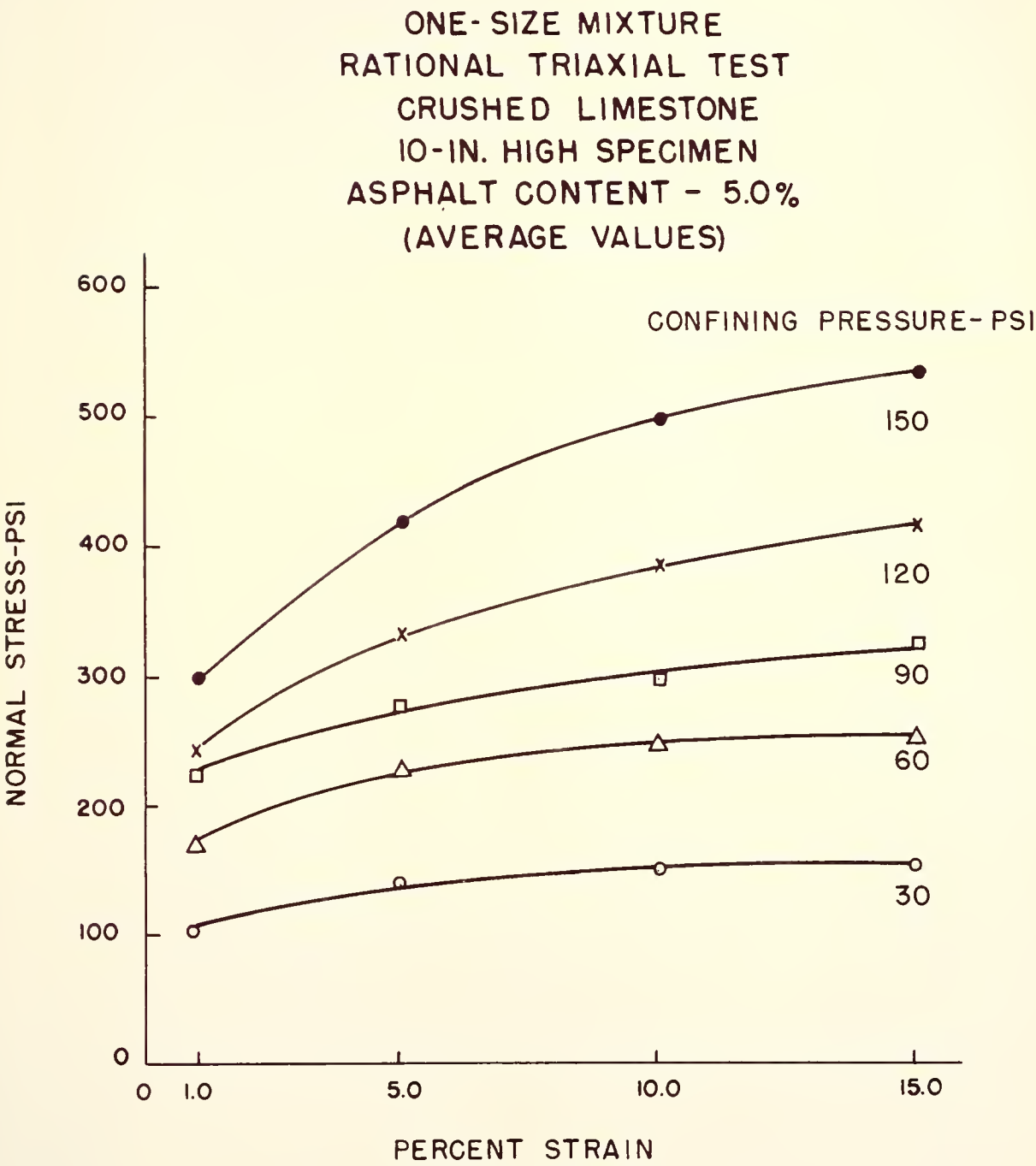


FIG. 23

confining pressures investigated were calculated and tabulated in Table 5. The graphical form of these data is presented in Fig. 24, where it may be deduced that at least 92 percent of the shearing strength was mobilized when the bituminous-aggregate mixture had experienced a strain or deformation of 10.0 percent. For the purposes of this laboratory investigation, the values of normal stress developed by the one-size mixture at 10.0 percent strain were considered as the practical values of shearing resistance for this paving mixture. This hypothesis was a logical compromise between the two important considerations, strength and deformation, for plastic materials.

Degree of Compaction and Specimen Height (Irrational Triaxial Test). The effects of the degree of compaction on the triaxial stability of the one-size mixture were investigated for crushed limestone specimens having an asphalt content of 5.0 percent by weight of the total mixture. The normal method of forming specimens was by the double-plunger compaction procedure. To obtain one-size specimens with density values greater than those produced by the regular double-plunger compaction method, this compaction procedure was modified to include the vibratory compaction of the specimen with a specially designed compaction foot attached to a pneumatic vibrator. The double-plunger method and the double-plunger with vibration method are outlined in detail under the heading, Preparation of the Test Specimen, in the

TABLE 5

Ratio of Normal Stress at 1.0, 5.0, and 10.0 Percent Strain to Normal Stress at 15.0 Percent Strain for Various Confining Pressure

Crushed Limestone — One-Size Mixture

Asphalt Content - 5.0% — 10-in. High Specimen

Confining Pressure psi	Ratio of Normal Stress at Various Percent Strain to Normal Stress at 15.0 Percent Strain			
	1.0	5.0	10.0	15.0
30	0.656	0.905	0.981	1.000
60	0.675	0.904	0.977	1.000
90	0.686	0.848	0.921	1.000
120	0.579	0.795	0.934	1.000
150	0.564	0.785	0.936	1.000

STRESS RATIO VS. CONFINING PRESSURE

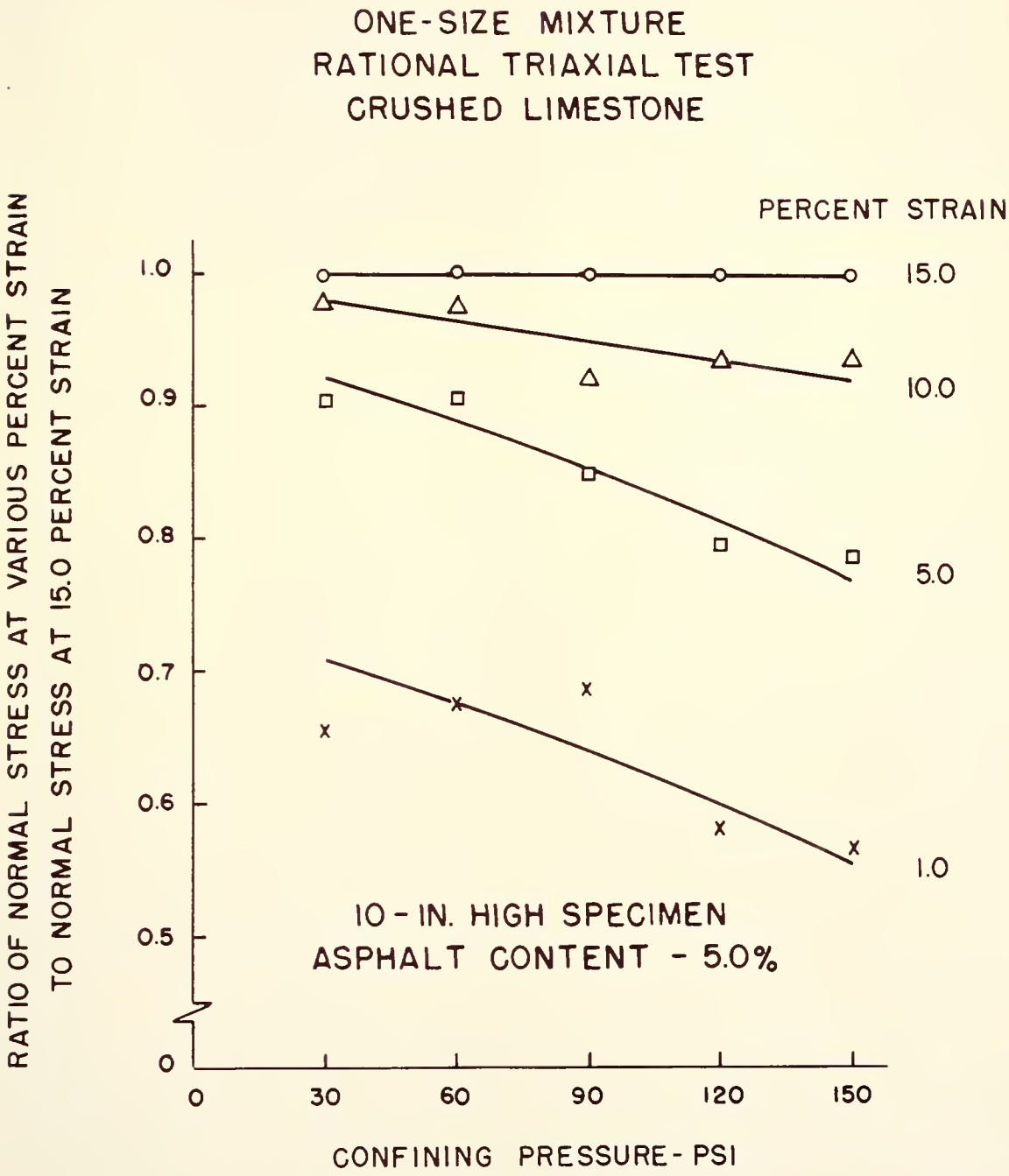


FIG. 24

section, Apparatus and Procedure. This vibrator with the compaction foot attached is pictured in Fig. 5. The results of this rational triaxial investigation are illustrated in Fig. 25 where the confining pressure is plotted against the normal stress at various strain values. These curves of the one-size mixture compacted to a greater density are analogous to the rational triaxial stability values obtained for the specimens formed at a lower density (see Fig. 22). Again the pattern of these curves followed the same sequence. The normal stresses at 1.0 percent strain increased at a decreasing rate (although the decreasing rate for the vibrated rational specimens was less than that for the non-vibrated test specimens) with an increase in the confining pressure, while the normal stress values at 5.0 percent strain showed a linear relationship with confining pressure. The values of normal stress at 10.0 and 15.0 percent strain increased at an increasing rate as higher confining pressures were applied to the rational triaxial specimens.

One apparent discrepancy was evident when a comparison of the non-vibrated specimen data (Fig. 22) was made with the vibrated specimen results (Fig. 25). The normal stresses at 30 and 60 psi confining pressures were higher for the non-vibrated specimens (lower densities) than for the vibrated specimens (higher densities), while the reverse was apparent for the shearing stresses at the 90, 120, and 150 psi confining pressures. The probable explanation for this heterodox

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

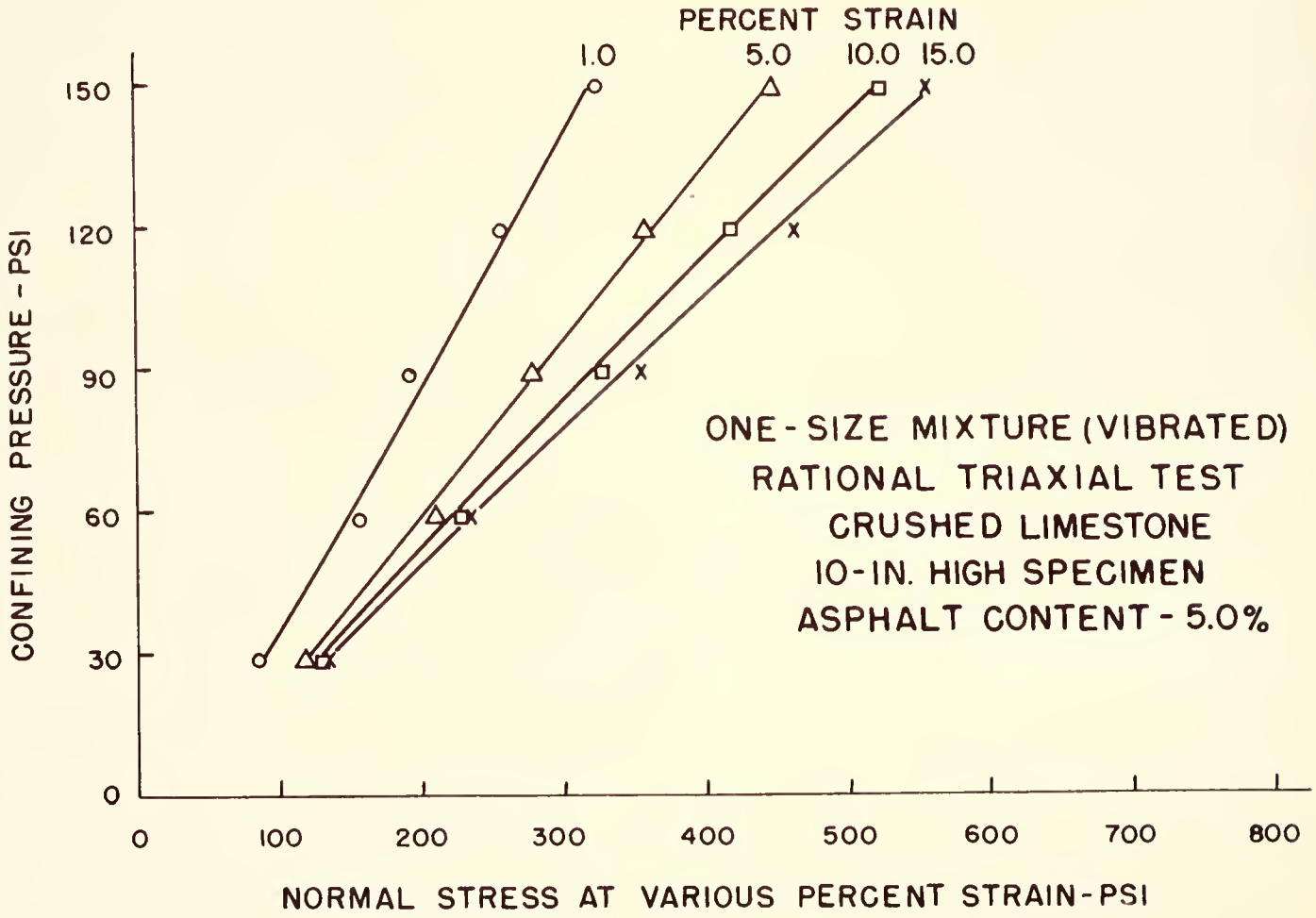


FIG. 25

condition is the interrelated and complicated interaction between confining stresses, rearrangement of the aggregate particles, and reduction in the volume of the triaxial test specimen during test.

The effects of specimen height on the triaxial strength of bituminous-aggregate mixtures tested at high confining pressures were investigated for both non-vibrated and vibrated specimens of this one-size mixture. The irrational triaxial test results for 4-in. high specimens are presented graphically in Fig. 26 for the double-plunger compaction method and in Fig. 27 for the double-plunger with vibration compaction procedure. The asphalt content was again maintained at 5.0 percent by weight of the total mixture. The pattern of curves representing the relationship between the confining pressure and the normal stress for the various strains is similar in each case to those representing the stability properties of the rational triaxial specimens. However, the irrational triaxial test data at 1.0 percent strain was best represented by a straight line for both degrees of compaction. At the three higher strain values of 5.0, 10.0, and 15.0 percent, the stress curves in both figures show that normal stress increased at an increasing rate as the higher confining pressure was reached.

A comparison of the rational test results as shown in Fig. 22 to the irrational test data in Fig. 26 for the non-vibrated specimens indicated that the normal stresses at 30

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

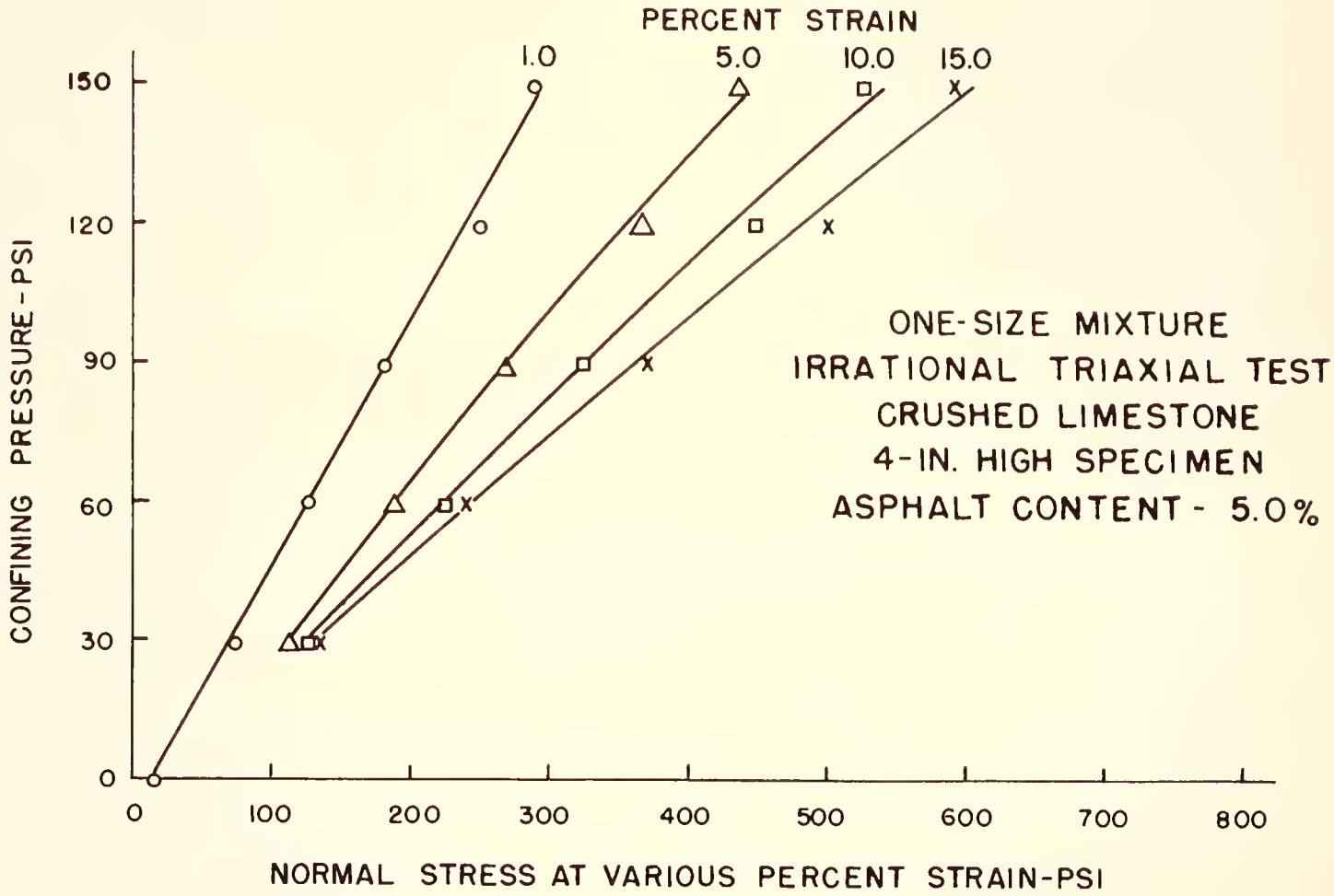


FIG. 26

CONFINING PRESSURE VS. NORMAL STRESS AT VARIOUS PERCENT STRAIN

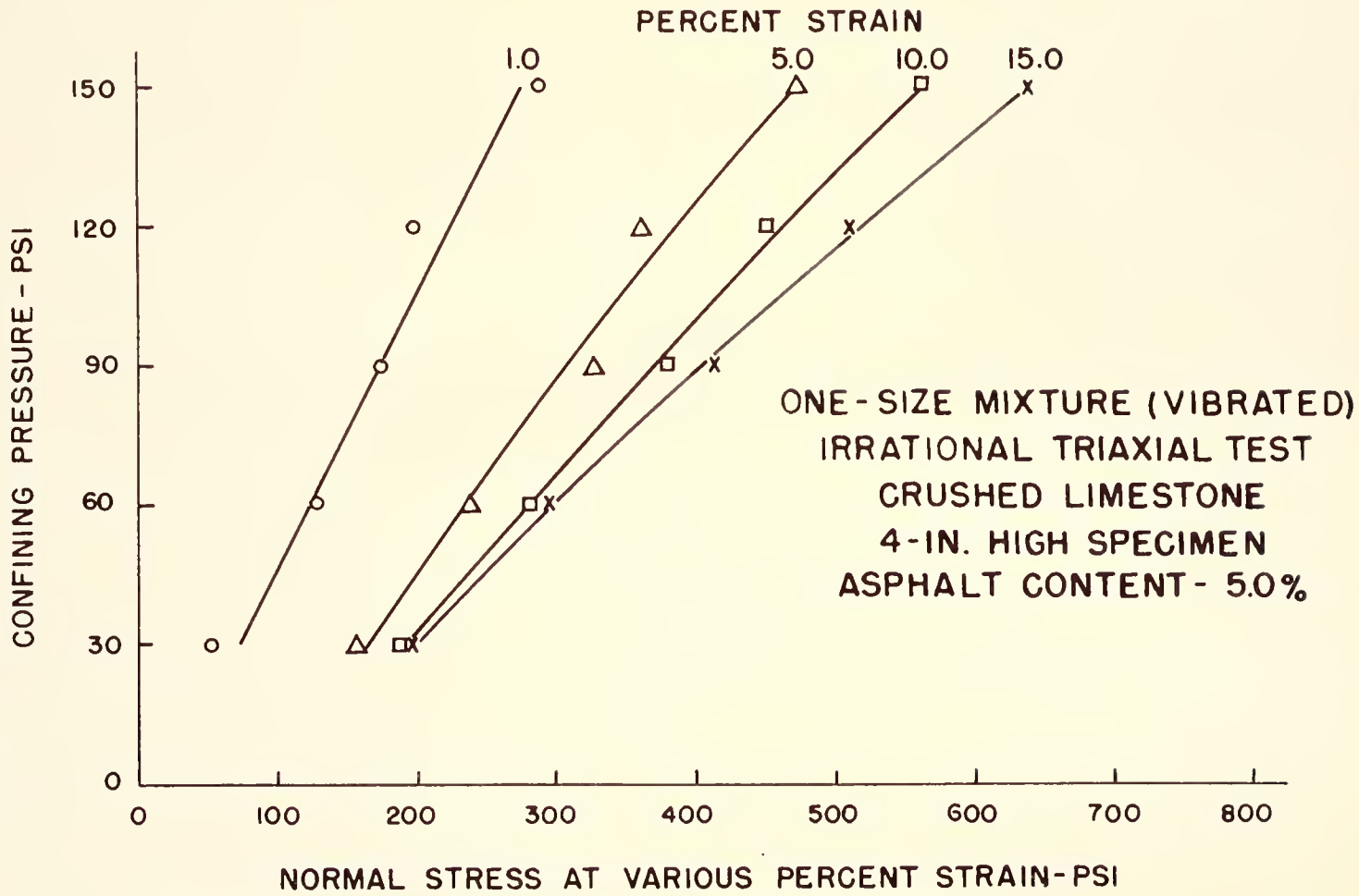


FIG. 27

and 60 psi confining pressure were higher for the rational specimens (10-in. high) than for the irrational specimens (4-in. high), while the reverse was true for the higher confining pressures of 90, 120, and 150 psi. This phenomenon was previously noted in the study of the non-vibrated and vibrated specimens having a height of 10 in.

An appraisal of the data for the irrational test specimens for the two degrees of compaction evidenced an increase in shearing strength for the more dense specimens (vibrated) as presented in Fig. 27 over the less dense specimens (non-vibrated) as illustrated in Fig. 26 at all but the 1.0 percent strain value. The same pattern was evident when the rational (Fig. 25) and irrational (Fig. 27) test results for the vibrated specimens were computed. The increase in the lateral support of the 4-in. high specimens due to the shearing stresses developed between the ends of the specimen and the loading plates produced higher strength values in the irrational triaxial test at the 5.0, 10.0, and 15.0 percent strain. However, the vibrated rational specimens possessed greater triaxial stability at a strain of 1.0 percent.

These discrepancies cannot be fully appraised as the stress properties of this one-size mixture are greatly influenced by the complicated interaction of lateral support, rearrangement of the aggregate particles, and reduction in the volume of the triaxial specimen during test. The curvilinear pattern of these relationships between

the confining pressure and the normal stress at various strains prevents the analysis of the stability properties in terms of such constant values as the cohesion and the angle of internal friction. Only the shearing stress of this mixture for any given condition of confining pressure and strain is a true representation of its strength.

Type of Aggregate. In evaluating the variable of aggregate type for rational triaxial testing at high confining pressures, 10-in. high specimens were formed from uncrushed gravel and natural sand at asphalt contents of 4.0 and 5.0 percent by weight of the total mixture. A comparison of the triaxial strengths at 10.0 percent strain of these specimens with the corresponding values obtained from crushed limestone specimens having identical grading and asphalt contents is presented in Fig. 28 for 30, 90, and 150 psi confining pressures. As developed earlier in this section, the failure criterion for the one-size mixture was established as a strain of 10.0 percent. The crushed limestone produced the greater shearing strengths for the two one-size mixtures throughout the range of confining pressures. This was attributed to the larger degree of interlocking among these angular aggregate particles than occurred with the uncrushed gravel and natural sand.

Asphalt Content. The effects of high confining pressure on the triaxial stability were determined for the one-size mixture having asphalt contents of 3.0, 4.0, and 5.0 percent by weight of the total mixture. The results of the rational

CONFINING PRESSURE VS. NORMAL STRESS AT 10.0 PERCENT STRAIN

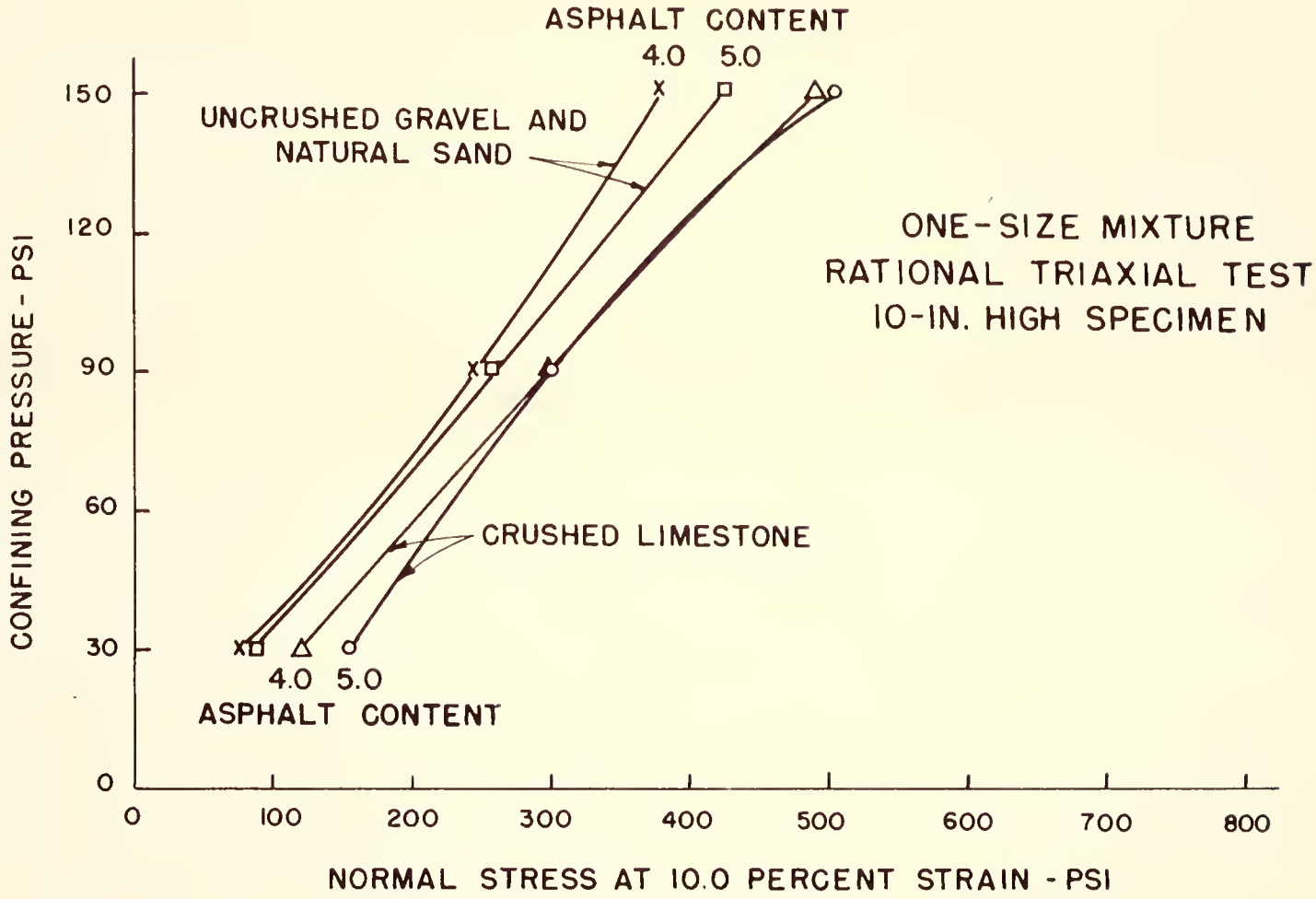


FIG. 28

triaxial tests for the low and high confining pressures of 30 and 150 psi, respectively, are presented in Fig. 29 for the crushed limestone specimens. The similar case for the uncrushed gravel and natural sand specimens is shown in Fig. 30. For the failure criterion of 10.0 percent strain, these figures exhibit the common premise that the confining pressure has little or no effect on the determination of asphalt content for maximum strength for both types of aggregate. The maximum triaxial stability was reached for an asphalt content near 5.0 percent by weight of the total mixture for both the crushed limestone and the uncrushed gravel and natural sand mixtures of the one-size grading.

Stabilometer Test. In the Stabilometer test analysis of the strength properties of the one-size mixture made from these two aggregate types and for the same range of asphalt content as for the triaxial tests, a trend that was opposite to the rational triaxial results was indicated. This is shown in Fig. 31. Although the Stabilometer stability values are exceedingly low, the results further substantiate the greater strength that the crushed limestone mixture has in comparison to the uncrushed gravel and natural sand. However, the curves in Fig. 31 illustrate that the asphalt content giving maximum strength was near 3.0 percent by weight of the total mixture for both types of aggregate. The probable explanation for the low stability values and the low optimum asphalt content shown by the Stabilometer results lies in the

NORMAL STRESS AT 10.0 PERCENT STRAIN VS. ASPHALT CONTENT

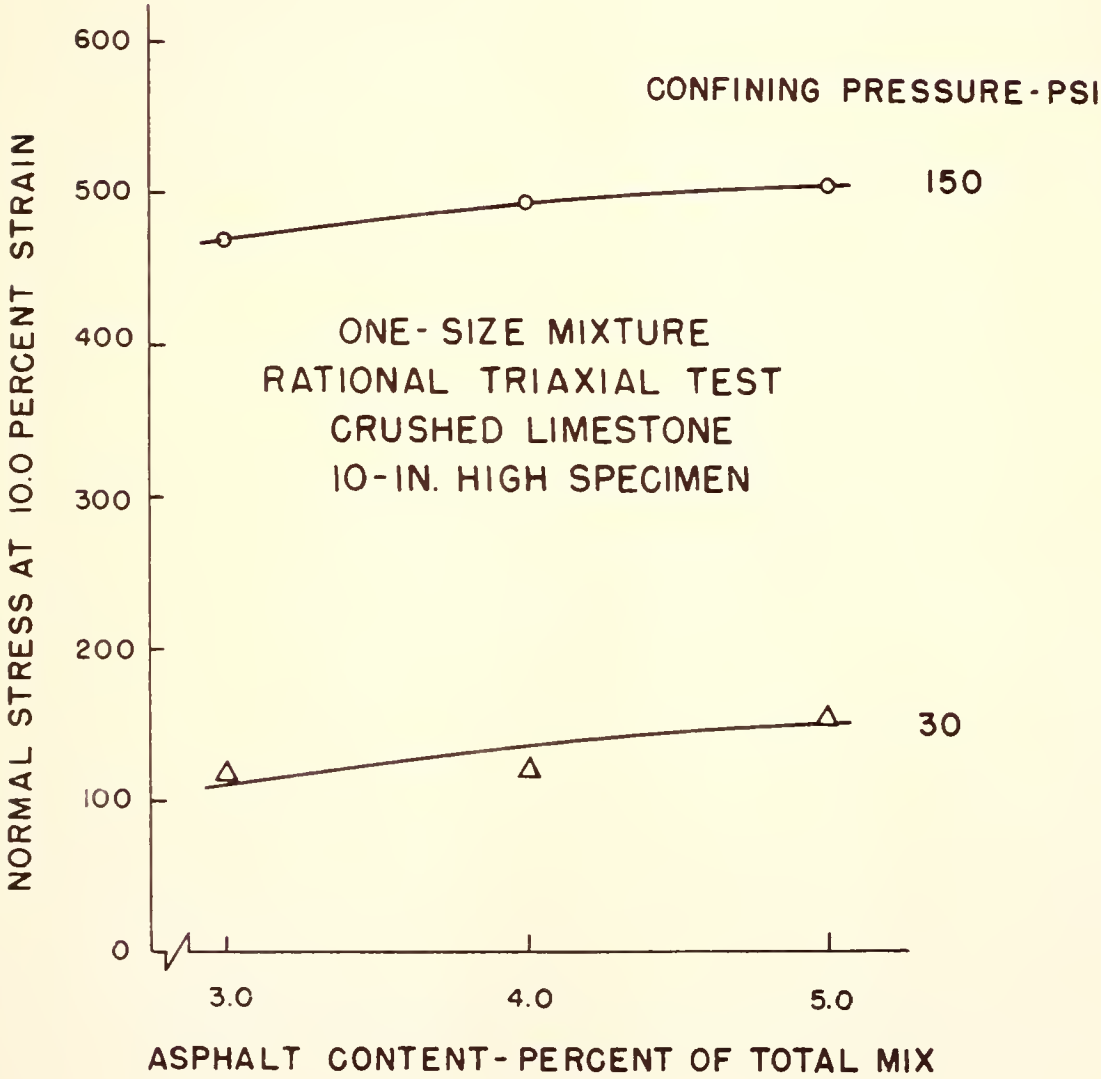


FIG. 29

NORMAL STRESS AT 10.0 PERCENT STRAIN VS. ASPHALT CONTENT

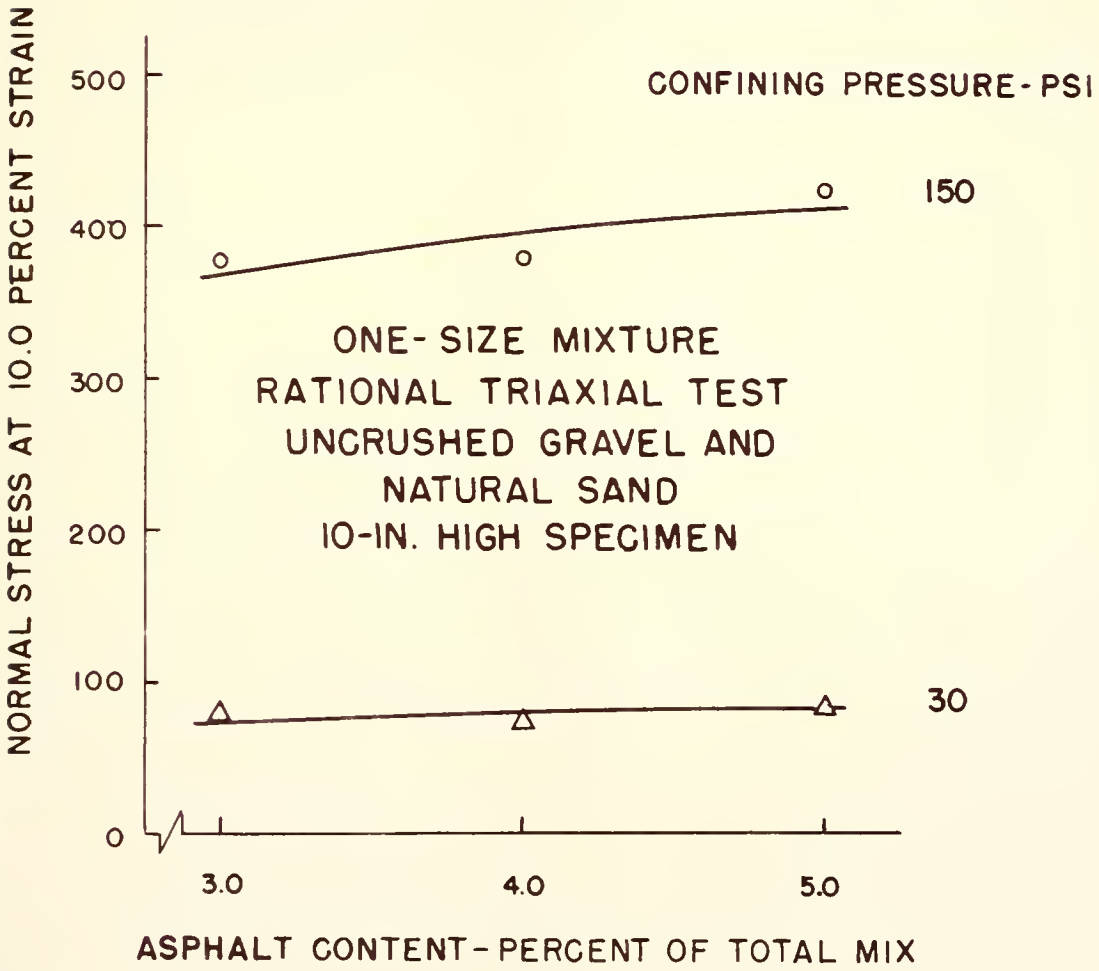


FIG. 30

STABILOMETER STABILITY VS. ASPHALT CONTENT

ONE-SIZE MIXTURE

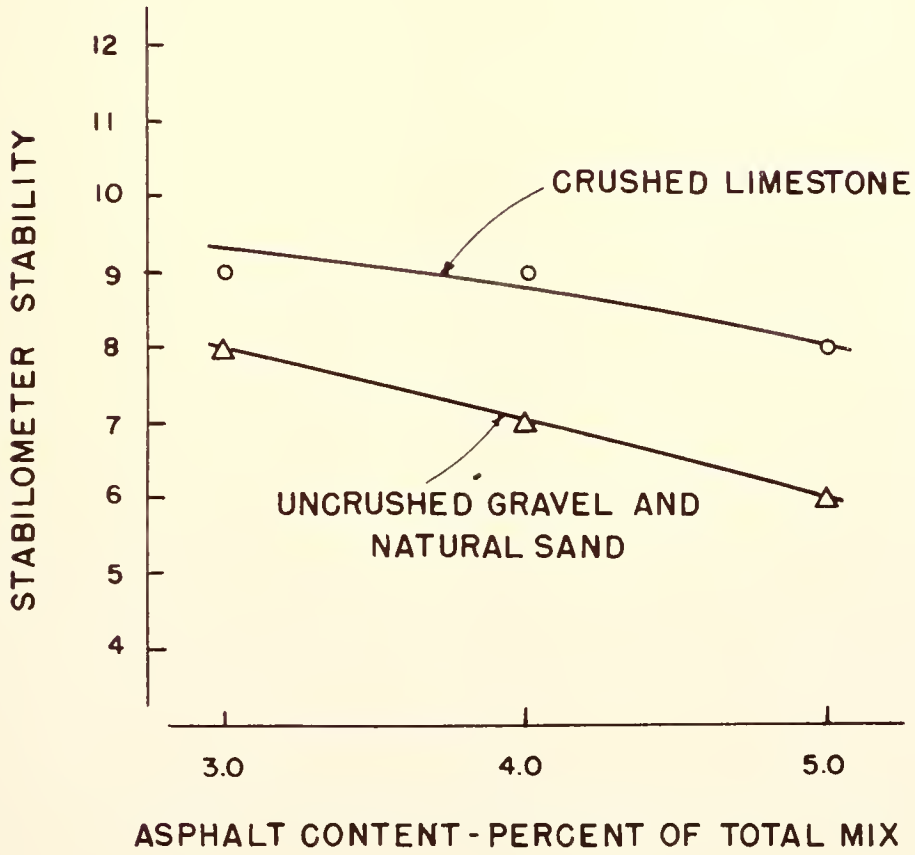


FIG. 31

fact that the Stabilometer test does not provide for sufficient specimen deformation to mobilize a substantial portion of the potential shearing resistance of the bituminous-aggregate mass.

SUMMARY OF RESULTS AND CONCLUSIONS

The following conclusions are drawn from the results of this laboratory investigation. However, these are to be considered valid only for the bituminous-aggregate mixtures of the same materials and gradings and the designated testing methods that were utilized in this study. Although field performance data are available for the mixture types used in this study, these conclusions have not been substantiated by field studies of the specific paving mixtures utilized, and their practical significance is limited in this respect.

1. The stress-strain curves representing the open-graded mixture were considerably different from those for the one-size mixture. The stress-strain diagrams for the open-graded mixture reached a peak normal stress value at a relatively low strain, while the normal stress continued to increase at a decreasing rate with an increase in the strain value for the one-size mixture.

2. When the open-graded mixture was evaluated by the rational triaxial test at high confining pressures, the shearing strength developed was accurately predicted by the Coulomb equation and represented by the constant parameters, cohesion and angle of internal friction.

3. The irrational triaxial tests on this open-graded mixture produced a linear Mohr rupture envelope for the range of confining pressures investigated. However, the shearing strength values were higher than those obtained by the

rational procedures because additional lateral support was afforded the short test specimens by the shearing stresses developed between the ends of the compacted specimen and the loading plates. Also, the slopes of these linear stress relationships of the rational and the irrational triaxial tests on the open-graded mixture were not the same, thus indicating that different values of cohesion and angle of internal friction were produced by these two testing procedures.

4. Low values of shearing stress were developed by the one-size mixture at low strain values and low confining pressures. However, when the specimen deformation and/or the confining pressure were increased, sufficient shearing resistance was developed to indicate more realistic stability values for this paving mixture. Thus, this one-size mixture may be evaluated satisfactorily in the laboratory if sufficient lateral support and the proper degree of deformation are provided to enable this bituminous-aggregate mixture to mobilize a significant portion of its shearing resistance. For the range of confining pressures considered, at least 90 percent of the available shearing resistance is developed if the compacted specimen is deformed to a strain of 10.0 percent. The conventional methods of triaxial compression testing as applied to bituminous mixtures must be modified to truly ascertain the complex stress-development pattern of this paving material.

5. The individual measures of the effects of specimen height and degree of compaction on the triaxial stability of the one-size mixture cannot be wholly isolated because of the interaction of lateral support, rearrangement of the aggregate particles, and change in specimen volume during test.

6. The triaxial compression tests at the low, intermediate, and high confining pressures showed that the more angular aggregate, crushed limestone, produced a larger resistance to shear than the more rounded aggregate, uncrushed gravel and natural sand, for both the open-graded and one-size mixtures.

7. For both the open-graded and one-size paving mixtures, the asphalt content which gave maximum stability was independent of the confining pressure used (30 - 150 psi).

8. The correlation of the rational triaxial test results with the Stabilometer test results for the open-graded mixture was found to be fairly good with respect to both stability and optimum asphalt content for design. However, the results of these two testing methods showed poor agreement in the stability analysis and the selection of an optimum asphalt content when used to evaluate the one-size mixture. The results of the Stabilometer test were held in question because of the limited specimen deformation permitted in this testing procedure.

SUGGESTIONS FOR FURTHER RESEARCH

As this research investigation was being conducted, it became apparent that various phases in the area of bituminous-aggregate mixtures required evaluation by laboratory and field studies. As any one laboratory investigation cannot wholly cover the given problem and its related components, the following itemized suggestions are presented as possibilities for further research to augment the findings obtained in this study:

1. Since all of the work in this research project was on bituminous mixtures composed of an asphalt cement and either a crushed limestone or an uncrushed gravel and natural sand aggregate, it would be desirable to evaluate the effects of triaxial testing at high confining pressures on bituminous-aggregate mixtures containing other types and gradations of aggregates and different types of bituminous materials. An investigation of the triaxial stability of "cold-mixes" would extend the results of this project.

2. An accurate and relatively simple method of determining the bulk density of porous, compacted specimens is an important procedure that should be developed to augment bituminous mixture research.

3. The change in volume that the one-size specimens experienced during the triaxial compression test was determined by comparing the volumes of the specimen measured both before and after the test. A more vivid understanding of

this volume-change and aggregate-reorientation phenomena could possibly be obtained by measuring the volume change as the triaxial test progressed. This study is suggested as a research project.

4. A more realistic evaluation by the triaxial test method could be approached if an actual measure of the field lateral support provided this paving material was obtained. Although theoretical attempts have been proposed to measure these confining stresses, no field test information has been compiled to verify these empirical calculations.

5. In the final analysis, the performance of a bituminous-aggregate mixture in the field under the action of traffic and weather is the controlling factor in any design of a bituminous mixture. This would necessitate the establishment of a field testing program to correlate the in-service findings with the laboratory results.

6. The development of a bituminous-mixture design method utilizing the triaxial test (rational and/or irrational) would prove most beneficial to the paving engineer. This procedure should be correlated with field performance to duplicate such important in-service factors as the degree of lateral support and the deformation of the paving mixture due to construction and traffic compaction.

7. Although many testing procedures are available for the evaluation of bituminous-aggregate mixtures, no relationship has been established between the type of mixture and the

test(s) applicable for accurately appraising that mixture. This detailed study would greatly facilitate the design of bituminous mixtures for both laboratory and field operations.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Aldous, W. M., Herner, R. C., and Price, M. H., "The Load Transmission Test for Flexible Paving and Base Courses, Part II, Triaxial Test Data on Structural Properties of Granular Base Materials," Technical Development and Evaluation Center, Civil Aeronautics Administration, Technical Development Report No. 144, June, 1951.

2. American Society for Testing Materials, ASTM Standards, Part 3, Philadelphia, Pa., 1955.

3. Balmer, G., "A General Analytic Solution for Mohr's Envelope," Proceedings, American Society for Testing Materials, Vol. 52, 1952.

4. Barber, E. S., "Application of Triaxial Compression Test Results to the Calculation of Flexible Pavement Thickness," Proceedings, Highway Research Board, Vol. 26, 1946.

5. Barber, E. S., "Triaxial Compression Tests Applied to Flexible Pavement Design," Public Roads, Vol. 25, No. 1, Sept., 1947.

6. Calderon, Hector M., "A Possible New Approach to Develop a Mohr Diagram," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

7. Carpenter, C. A., Goode, J. F., and Peck, R. A., "An Improved Triaxial Cell for Testing Bituminous Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

8. Corps of Engineers, Airfield Paving Design, Flexible Pavements, Engineering Manual for Military Construction, Part XII, Chapter 2, July, 1951.

9. Endersby, V. A., "The Analytic Mechanical Testing of Bituminous Mixes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 11, 1940.

10. Endersby, V. A., "The History and Theory of Triaxial Testing, and the Preparation of Realistic Test Specimens - A Report of the Triaxial Institute," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

11. Endersby, V. A., and Vallerga, B. A., "Laboratory Compaction Methods and Their Effects on Mechanical Stability Tests for Asphaltic Pavements," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

12. Gibson, R. E., "Experimental Determination of the True Cohesion and True Angle of Internal Friction in Clays," Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, Session 2, Switzerland, August, 1953.

13. Goetz, W. H., "Comparison of Triaxial and Marshall Test Results," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

14. Goetz, W. H., and Chen, C. C., "Vacuum Triaxial Technique Applied to Bituminous-Aggregate Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 19, 1950.

15. Harris, F. A., "Selection and Design of Semi-Flexible and Conventional Type Pavements," Proceedings, Highway Research Board, Vol. 35, 1956.

16. Hennes, R. G., and Wong, C. C., "Physical Interpretation of Triaxial Test Data," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

17. Herrin, M., "Effects of Aggregate Shape on the Stability of Bituminous Mixes," A Thesis submitted to Purdue University for the degree of Doctor of Philosophy, May, 1954.

18. Highway Research Board, Bituminous Paving Mixtures, Fundamentals for Design, Bulletin 105, Washington, D. C., 1955.

19. Housel, W. S., "Interpretation of Triaxial Compression Tests on Granular Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 19, 1950.

20. Hveem, F. N., and Carmany, R. M., "The Factors Underlying the Rational Design of Pavements," Proceedings, Highway Research Board, Vol. 28, 1948.

21. Hveem, F. N., and Davis, H. E., "Some Concepts Concerning Triaxial Compression Testing of Asphaltic Paving Mixtures and Subgrade Materials," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

22. Hveem, F. N., and Vallerga, B. A., "Density Versus Stability," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

23. Lambe, T. W., Soil Testing for Engineers, John Wiley and Sons, Inc., New York, 1954.

24. Mack, C., "Deformation Mechanism and Bearing Strength of Bituminous Pavements," Proceedings, Highway Research Board, Vol. 33, 1954.

25. McCarty, L. E., "Correlation between Stability and Certain Physical Properties of Bituminous Materials," Proceedings, Highway Research Board, Vol. 33, 1954.

26. McCarty, L. E., and Hank, R. J., "A Comparison of Dimensions in the Mohr Diagram with Those in Other Diagrams Used in Stress Analysis," Proceedings, The Association of Asphalt Paving Technologists, Vol. 18, 1949.

27. McDowell, C. A., "Triaxial Testing: Its Adaptation and Application to Highway Materials, Part II - Use of the Triaxial Method in Road Design," Roads and Streets, March, 1950.

28. McDowell, C. A., "Triaxial Tests in Analysis of Flexible Pavements," Highway Research Board Research Report, 16-B, 33rd Annual Meeting, January 12-15, 1954.

29. McLeod, N. W., "A Rational Approach to the Design of Bituminous Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 19, 1950.

30. McLeod, N. W., "Application of Triaxial Testing to the Design of Bituminous Pavements," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

31. McLeod, N. W., "Influence of Tire Design on Pavement Design and Vehicle Mobility," Proceedings, Highway Research Board, Vol. 31, 1952.

32. McLeod, N. W., "Rational Design of Bituminous Paving Mixtures with Curved Mohr Envelopes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

33. McLeod, N. W., "The Design of Bituminous Mixtures with Curved Mohr Envelopes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

34. McLeod, N. W., "The Rational Design of Bituminous Paving Mixtures," Proceedings, Highway Research Board, Vol. 29, 1949.

35. McLeod, N. W., "The Stability of Granular and Cohesive Materials in Triaxial Compression," Proceedings, The Association of Asphalt Paving Technologists, Vol. 17, 1948.

36. Monismith, C. L., and Vallerga, B. A., "Relationship Between Density and Stability of Asphaltic Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 25, 1956.

37. Neppe, S. L., "Mechanical Stability of Bituminous Mixtures, A Summary of the Literature," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

38. Nijboer, L. W., "The Determination of Plastic Properties of Bitumen-Aggregate Mixtures and the Influence of Variations in the Composition of the Mix," Proceedings, The Association of Asphalt Paving Technologists, Vol. 16, 1947.

39. Smith, V. R., "Application of the Triaxial Test to Bituminous Mixtures - California Research Corporation Method," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1951.

40. Smith, V. R., "Triaxial Stability Method for Flexible Pavement Design," Proceedings, The Association of Asphalt Paving Technologists, Vol. 18, 1949.

41. Stanton, T. E., and Hveem, F. N., "Role of the Laboratory in the Preliminary Investigation and Control of Materials for Low Cost Bituminous Pavements," Proceedings, Highway Research Board, Vol. 14, Part II, 1934.

42. State Highway Commission of Indiana, Standard Specifications for Road and Bridge Construction and Maintenance, 1952.

43. Stevens, D. E., "Fundamentals of Stability Testing of Asphalt Mixes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

44. Taylor, D. W., Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1954.

45. Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, 1954.

46. The Asphalt Institute, Manual on Hot-Mix Asphaltic Concrete Paving, 1954.

47. The Asphalt Institute, Mix Design Methods for Hot-Mix Asphalt Paving, Manual Series No. 2, College Park, Md., 1956.

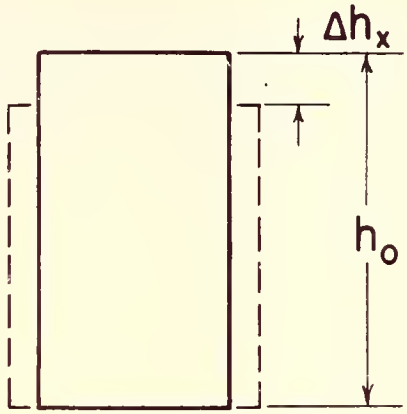
48. Vallerga, B. A., "The Triaxial Institute and the Stabilometer," Proceedings, Highway Research Board, Vol. 34, 1955.

49. Worley, H. F., "Triaxial Testing Methods Usable in Flexible Pavement Design," Proceedings, Highway Research Board, Vol. 23, 1943.

APPENDIX

APPENDIX A

DERIVATION OF RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING NO VOLUME CHANGE



ASSUMPTION: AT ALL TIMES THE TEST SPECIMEN IS CYLINDRICAL

WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA

Δh_x = CHANGE IN SPECIMEN HEIGHT

A_o = INITIAL CROSS-SECTIONAL AREA

h_o = INITIAL SPECIMEN HEIGHT

V_f = FINAL SPECIMEN VOLUME

V_o = INITIAL SPECIMEN VOLUME

DERIVATION: $V_f = A_x(h_o - \Delta h_x)$

$V_o = A_o h_o$

$V_o = V_f$ (NO VOLUME CHANGE)

$A_x(h_o - \Delta h_x) = A_o h_o$

$$A_x = \frac{A_o h_o}{(h_o - \Delta h_x)}$$

$$A_x = \frac{A_o}{\left(1 - \frac{\Delta h_x}{h_o}\right)}$$

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING NO VOLUME CHANGE

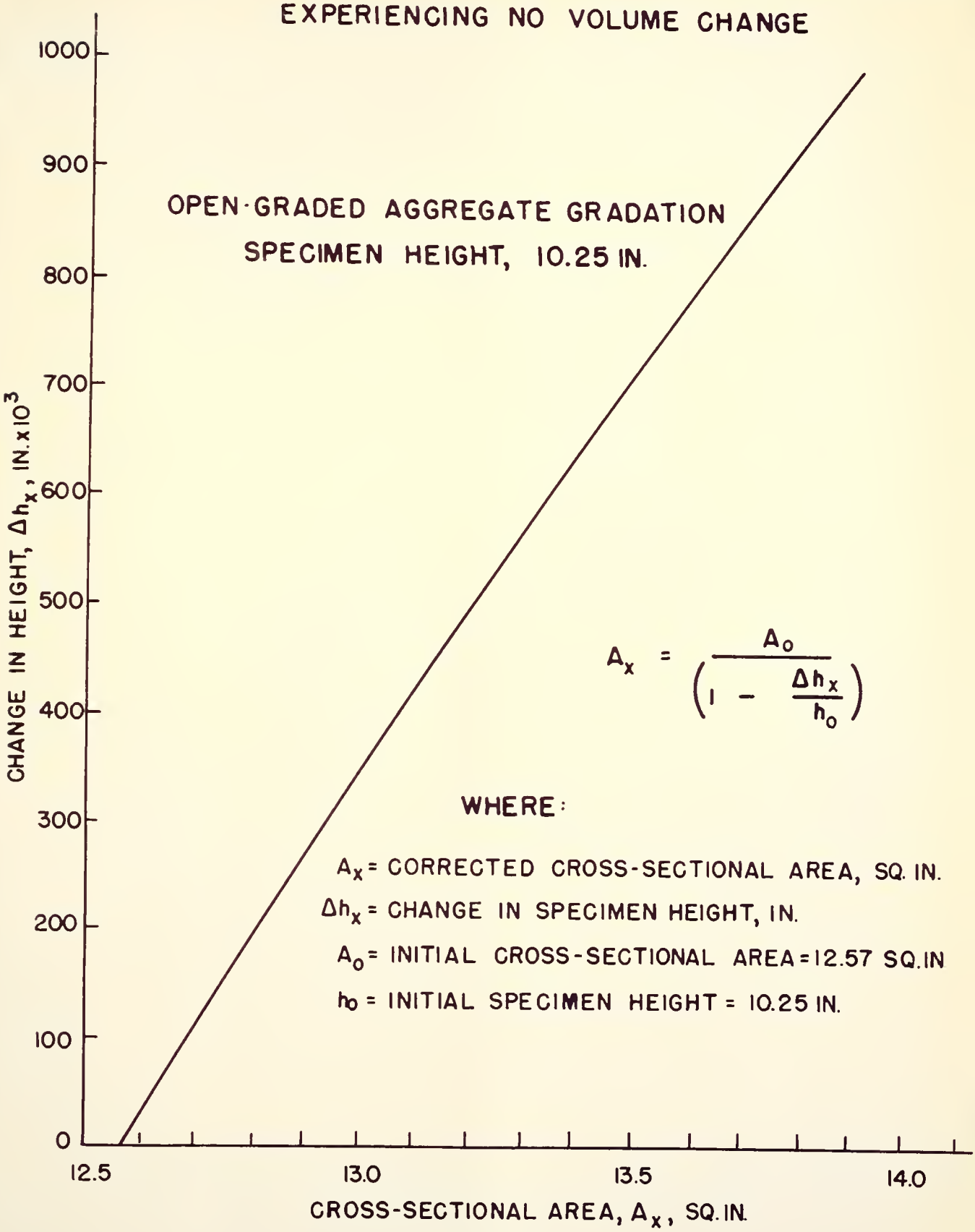


FIG. 32

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING NO VOLUME CHANGE

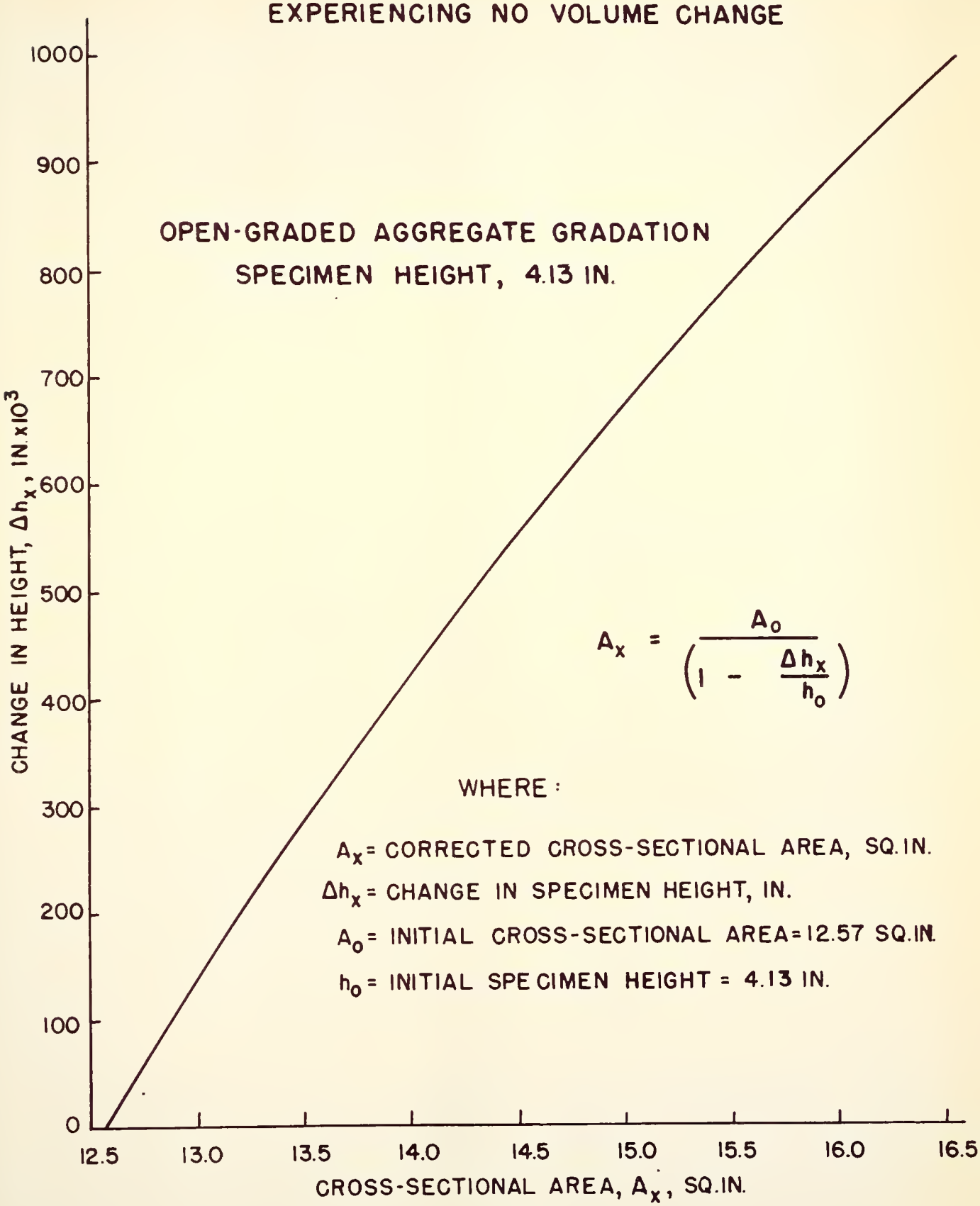
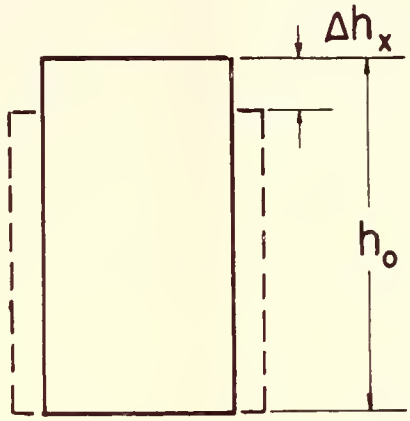


FIG. 33

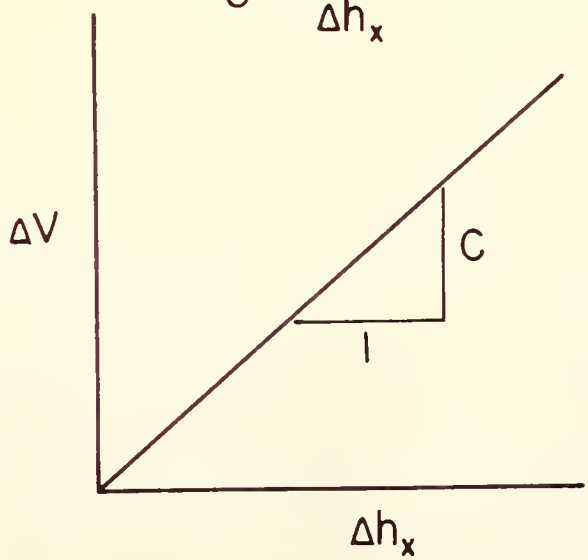
APPENDIX B

DERIVATION OF RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION



ASSUMPTION: AT ALL TIMES THE TEST SPECIMEN IS CYLINDRICAL

$$C = \frac{\Delta V}{\Delta h_x}$$



ASSUMPTION: THE CHANGE IN VOLUME IS PROPORTIONAL TO THE CHANGE IN HEIGHT

WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA

Δh_x = CHANGE IN SPECIMEN HEIGHT

A_o = INITIAL CROSS-SECTIONAL AREA

h_o = INITIAL SPECIMEN HEIGHT

V_f = FINAL SPECIMEN VOLUME

V_o = INITIAL SPECIMEN VOLUME

ΔV = CHANGE IN SPECIMEN VOLUME

C = VOLUME CHANGE PER CHANGE IN HEIGHT

DERIVATION:

$$V_f = A_x (h_o - \Delta h_x)$$

$$V_o = A_o h_o$$

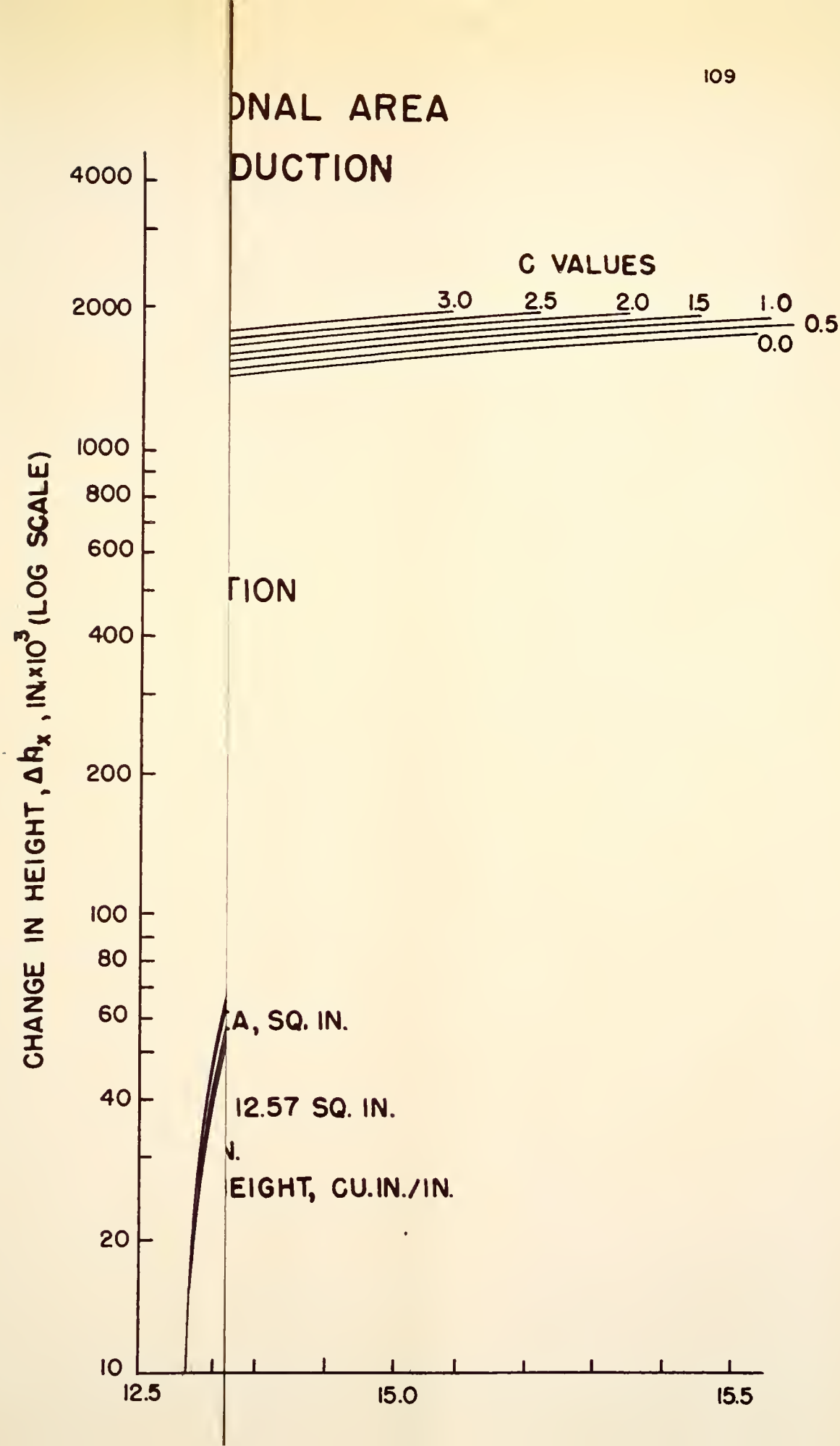
$$C = \frac{\Delta V}{\Delta h_x}$$

$$V_f = V_o - \Delta V \quad \text{(VOLUME REDUCTION)}$$

$$A_x (h_o - \Delta h_x) = A_o h_o - C \Delta h_x$$

$$A_x = \left(\frac{A_o h_o}{h_o - \Delta h_x}\right)\left(1 - \frac{C \Delta h_x}{A_o h_o}\right)$$

$$A_x = \left(\frac{A_o}{1 - \frac{\Delta h_x}{h_o}}\right)\left(1 - \frac{C \Delta h_x}{A_o h_o}\right)$$


ONAL AREA
DUCTION
C VALUES
3.0
2.5
2.0
1.5
1.0
0.5
0.0
TION
A, SQ. IN.
12.57 SQ. IN.
N.
EIGHT, CU.IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.x10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
15.0
15.5

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION

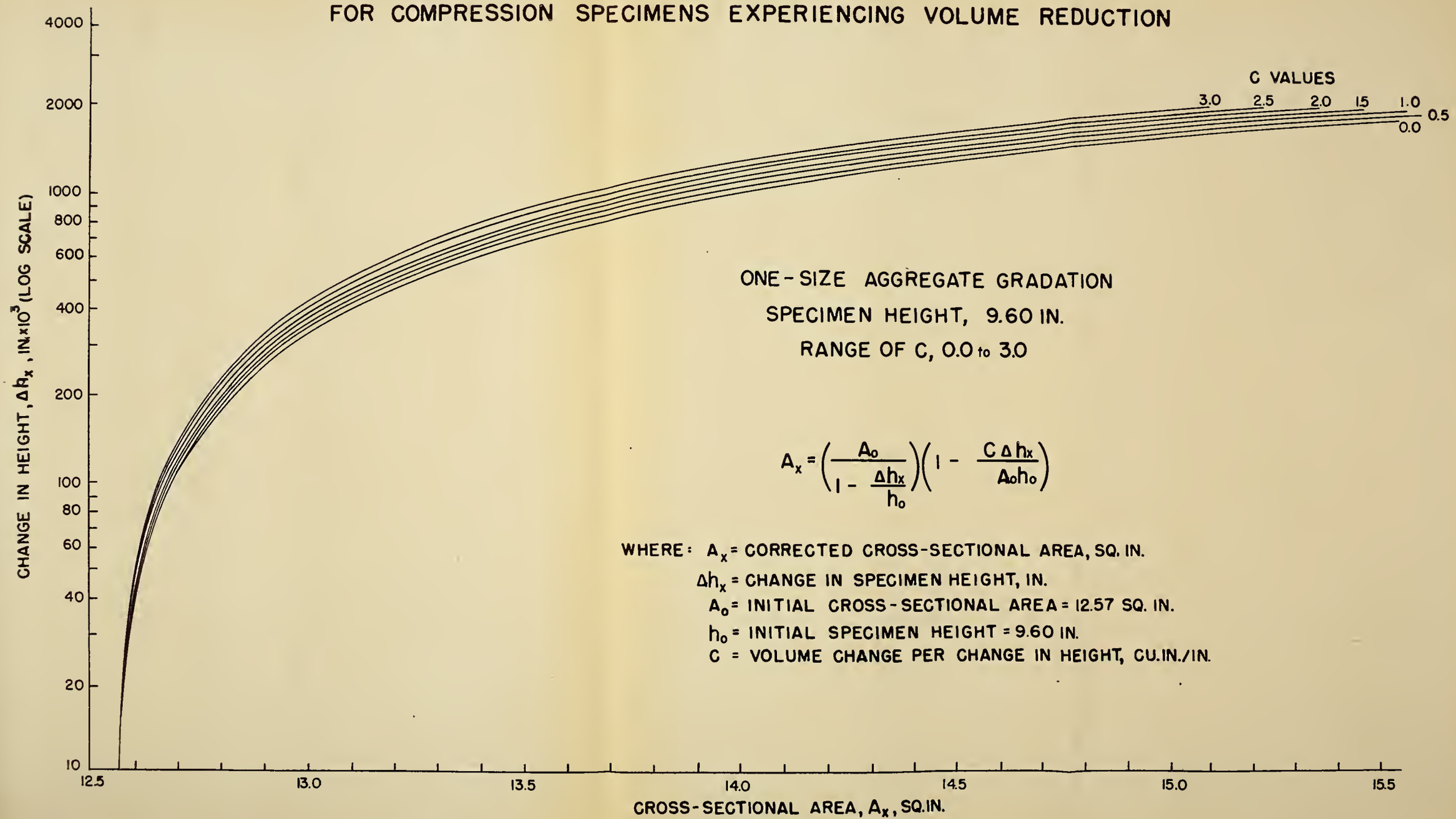
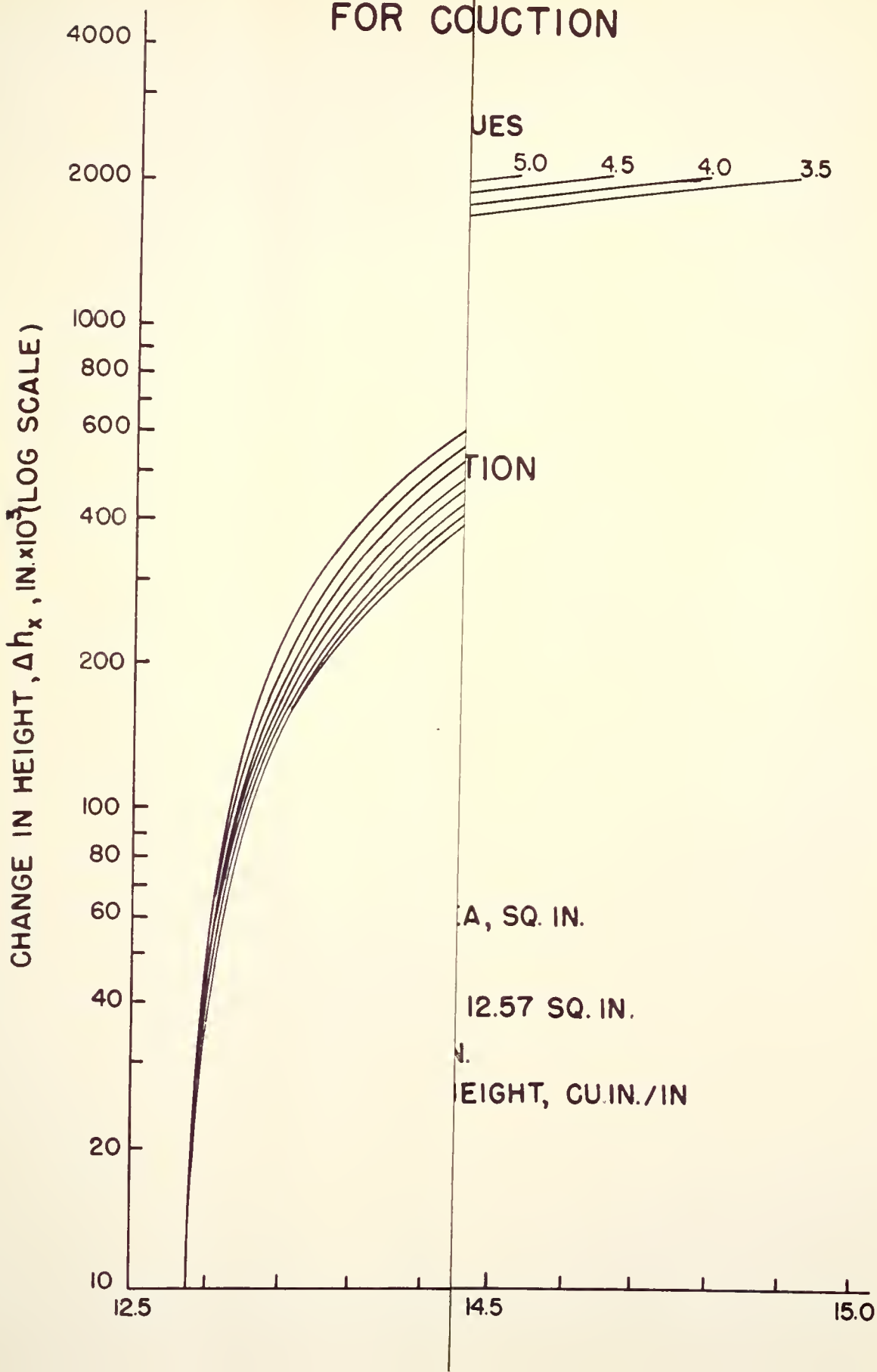


FIG. 34

RELATIONONAL AREA FOR COUCTION



RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION

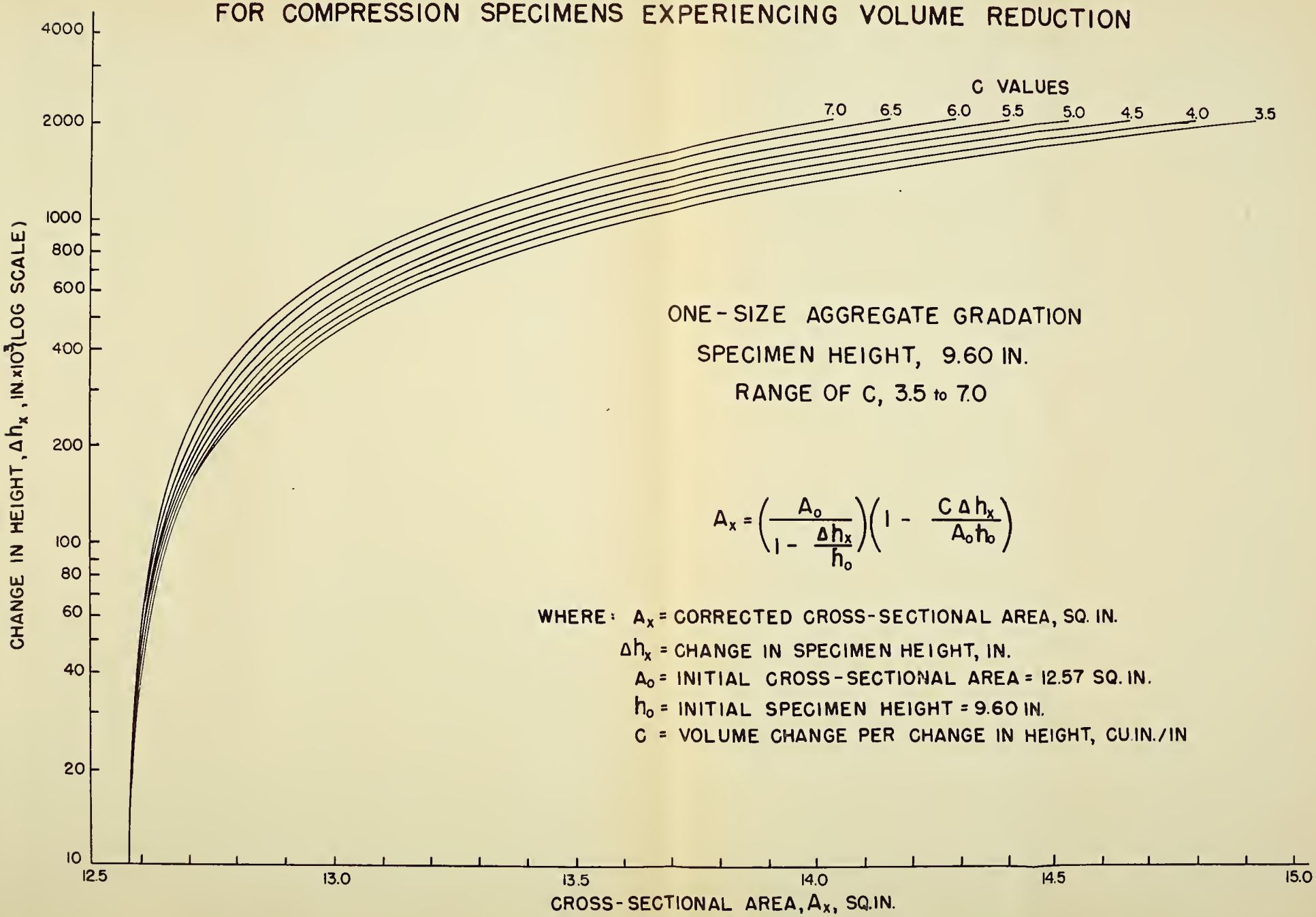
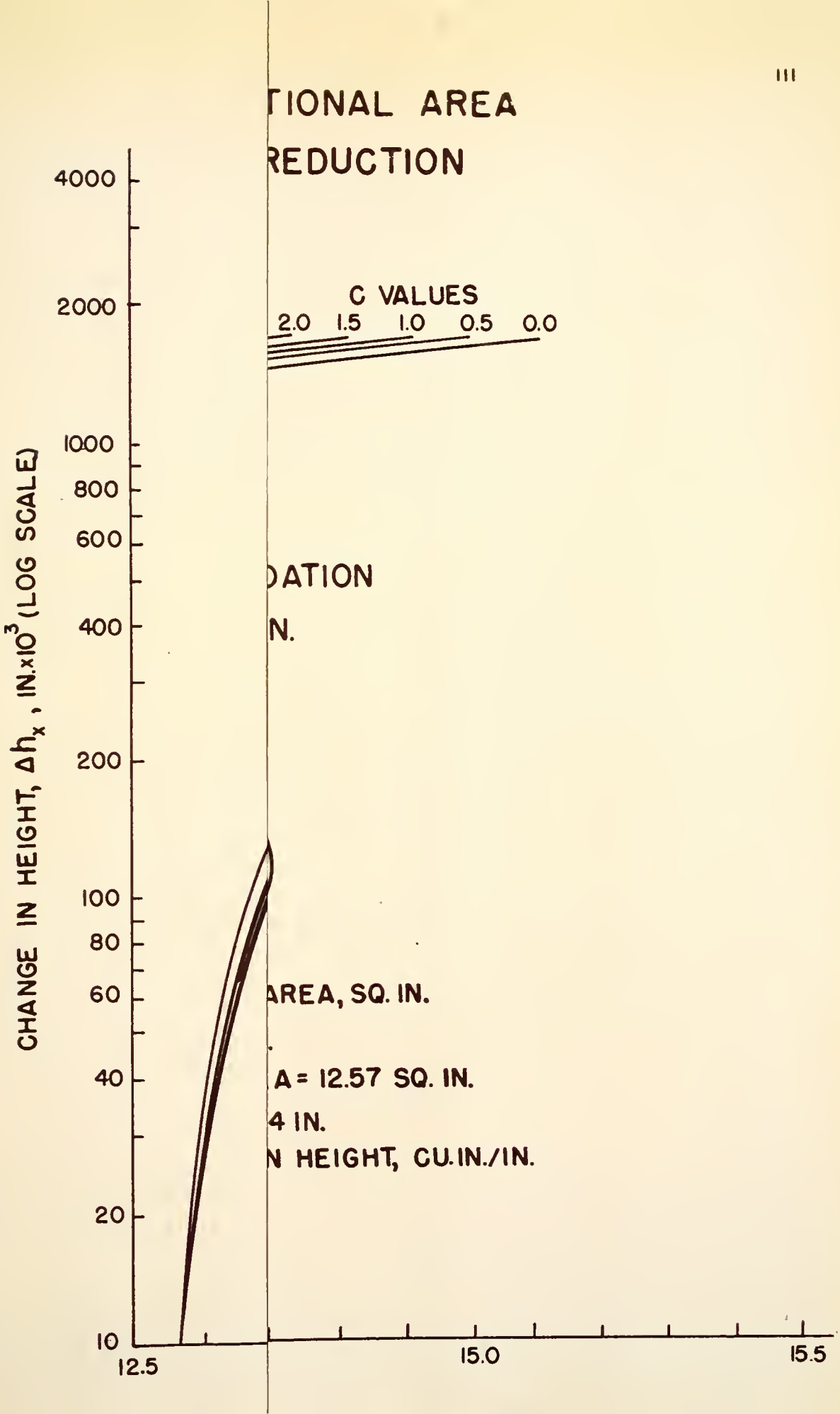


FIG. 35


TIONAL AREA
REDUCTION
C VALUES
2.0 1.5 1.0 0.5 0.0
DATION
N.
AREA, SQ. IN.
A = 12.57 SQ. IN.
4 IN.
N HEIGHT, CU.IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.x10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
15.0
15.5

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION

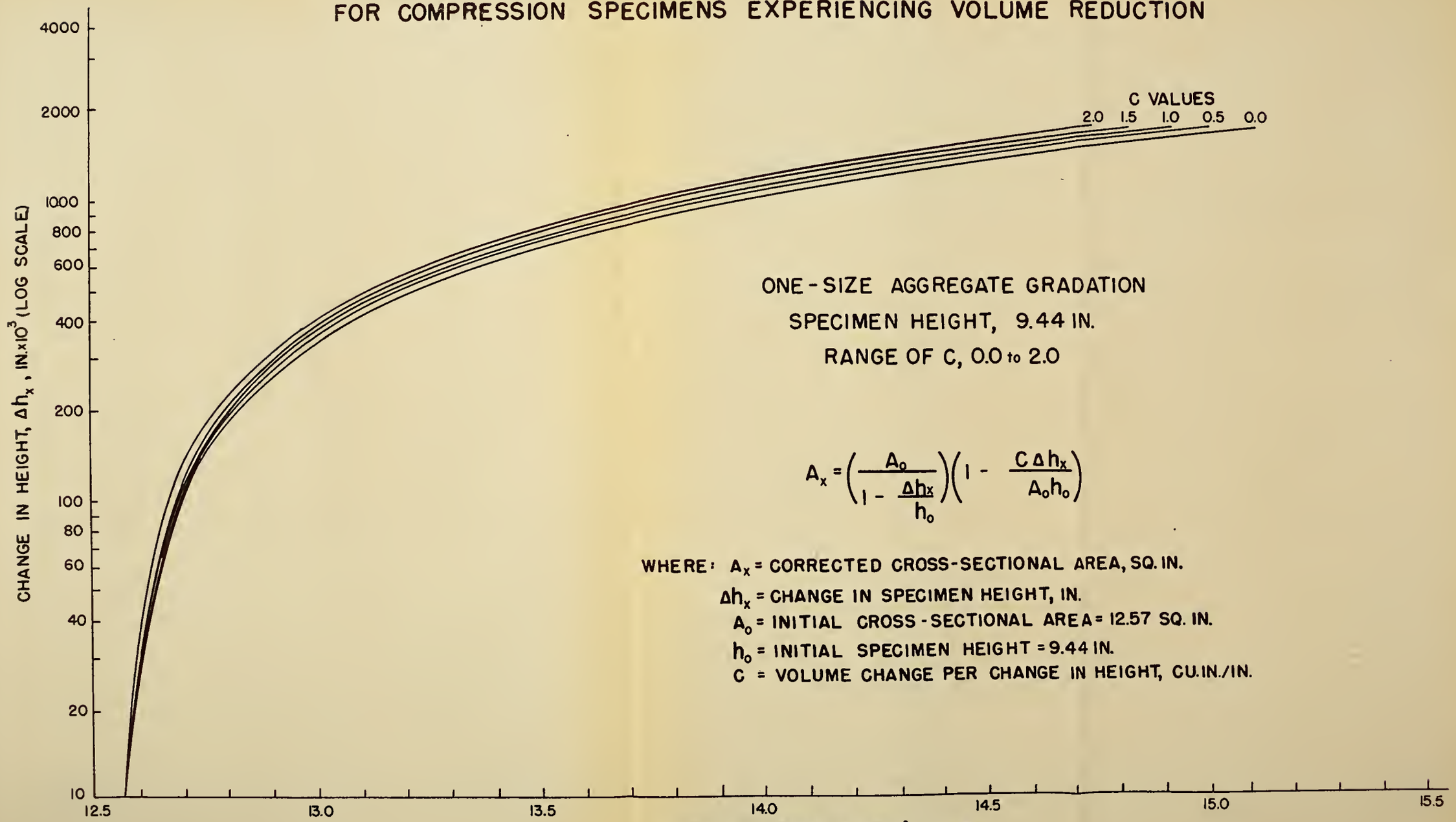
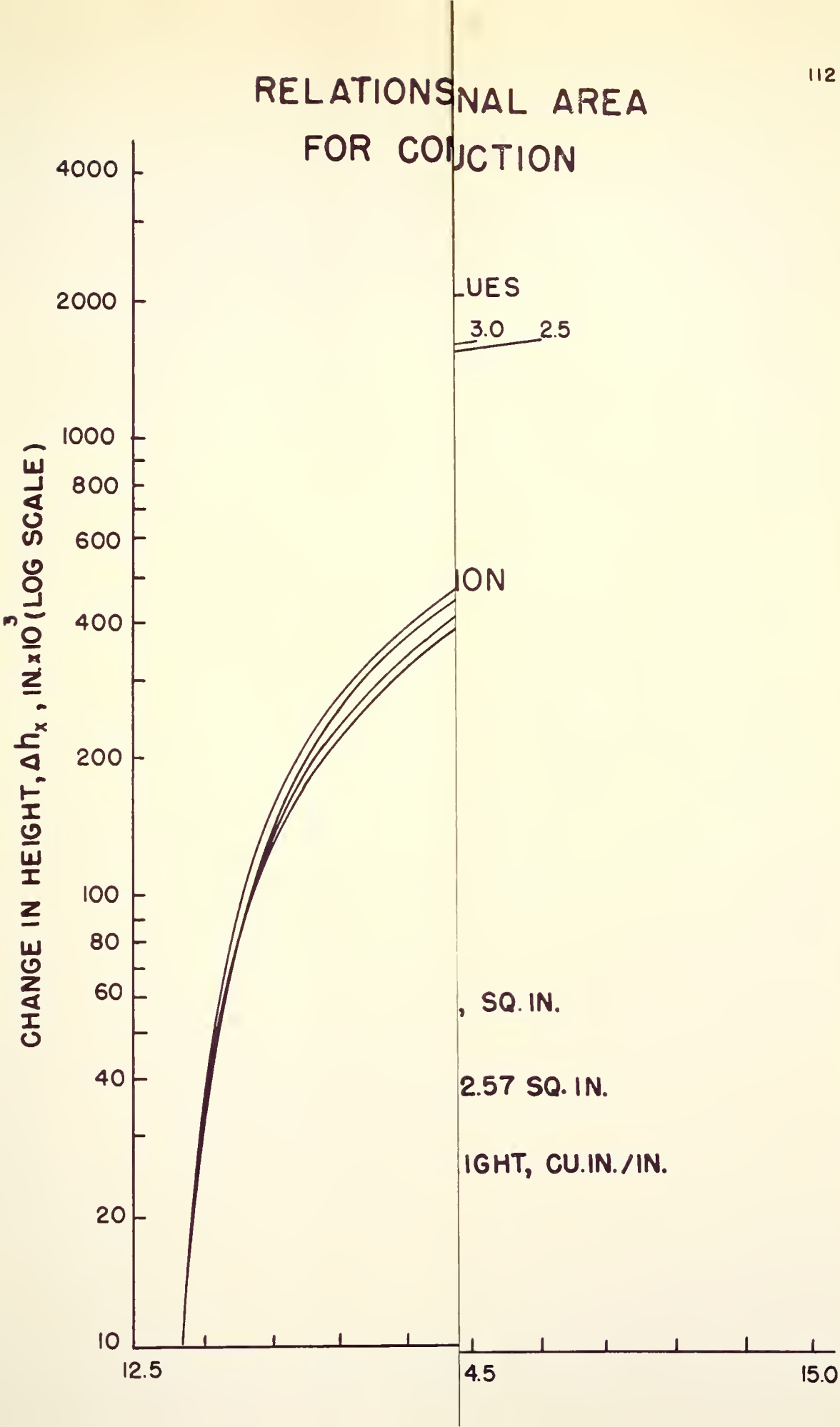


FIG. 36


RELATIONSNAL AREA
FOR COUCTION
LUES
3.0
2.5
ION
, SQ. IN.
2.57 SQ. IN.
IGHT, CU.IN./IN.
CHANGE IN HEIGHT, Δh_x, IN. x 10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
4.5
15.0

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION

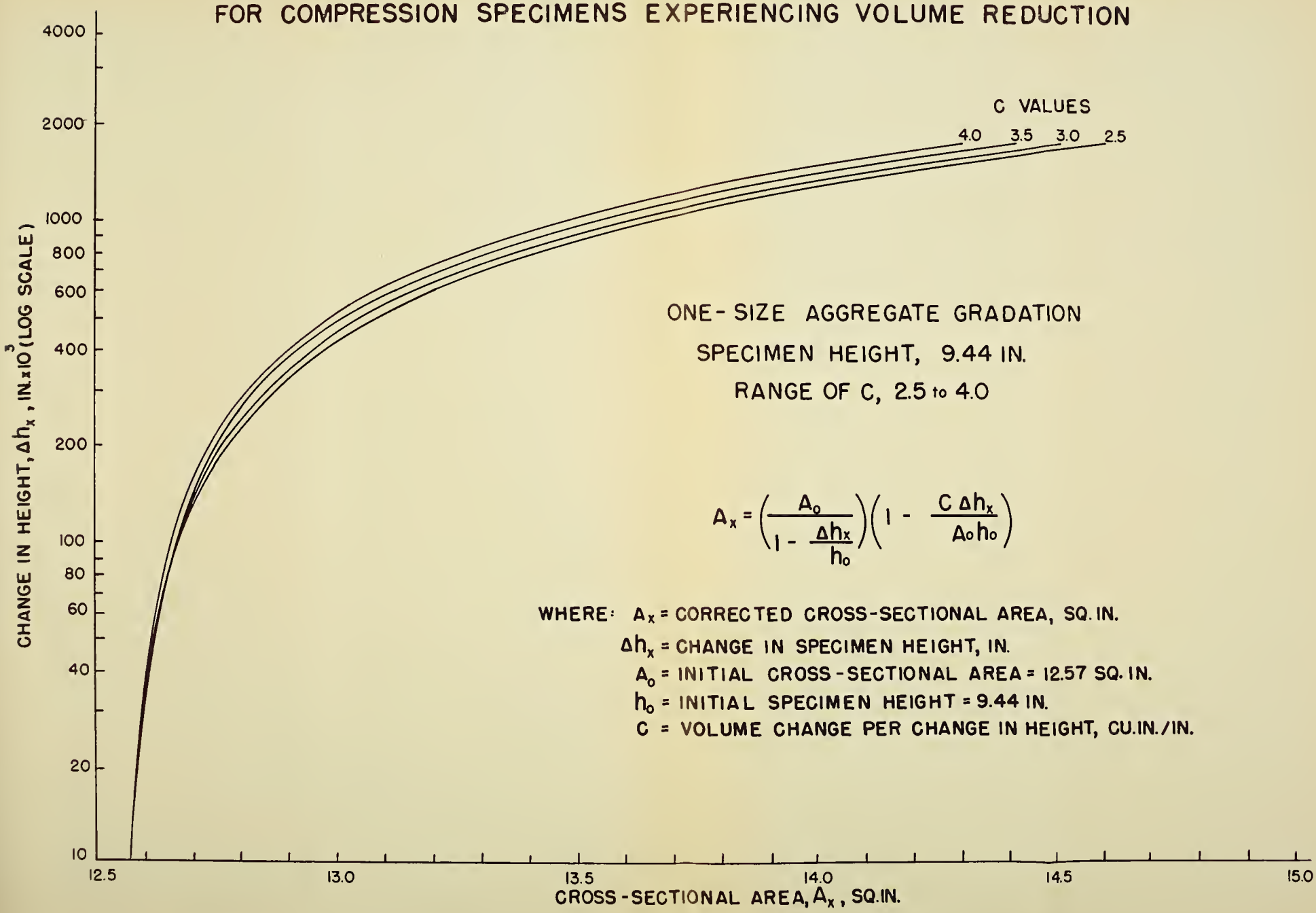
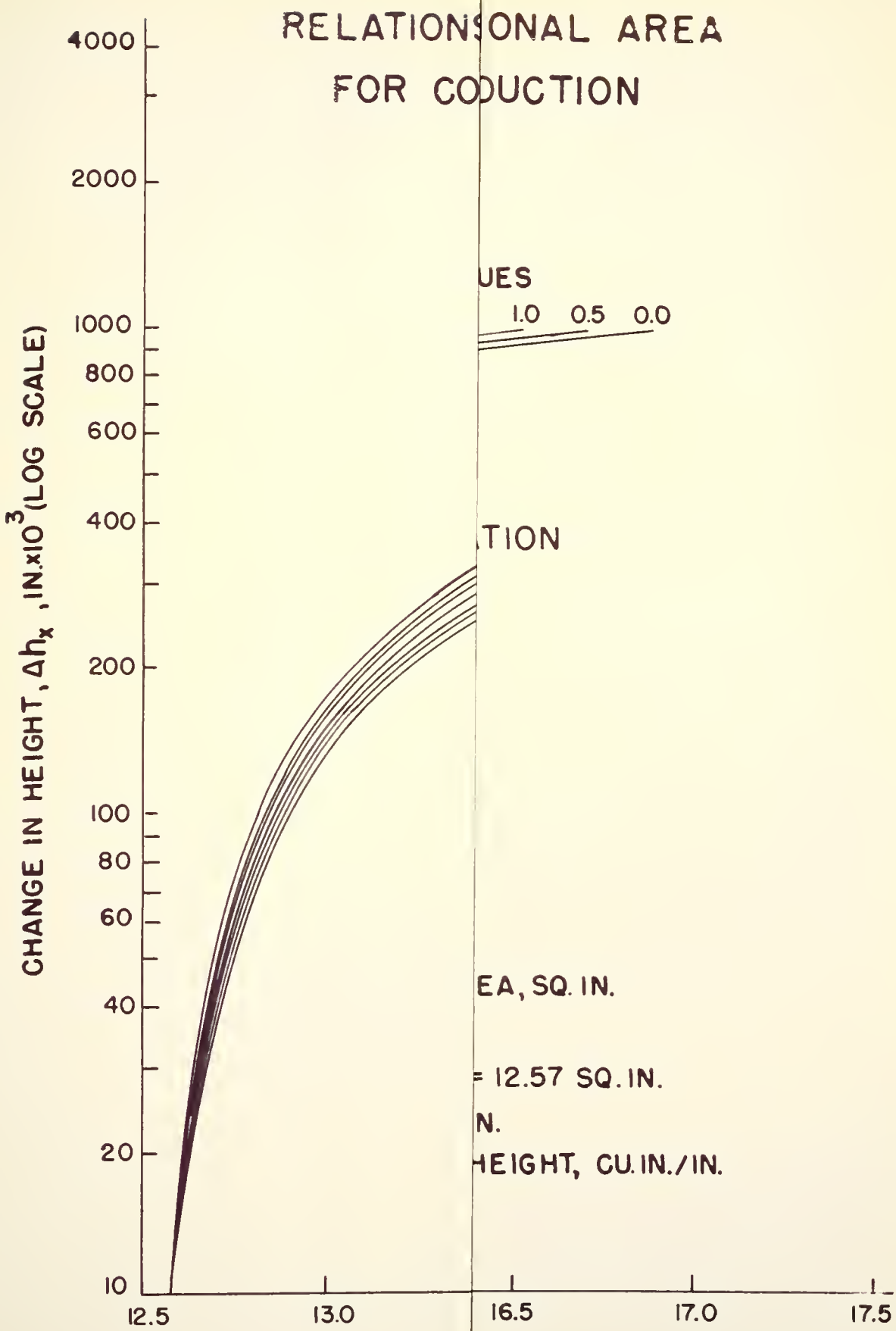


FIG. 37


RELATIONSONAL AREA
FOR COUCTION
UES
1.0
0.5
0.0
TION
EA, SQ. IN.
= 12.57 SQ. IN.
N.
HEIGHT, CU. IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.x10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
13.0
16.5
17.0
17.5

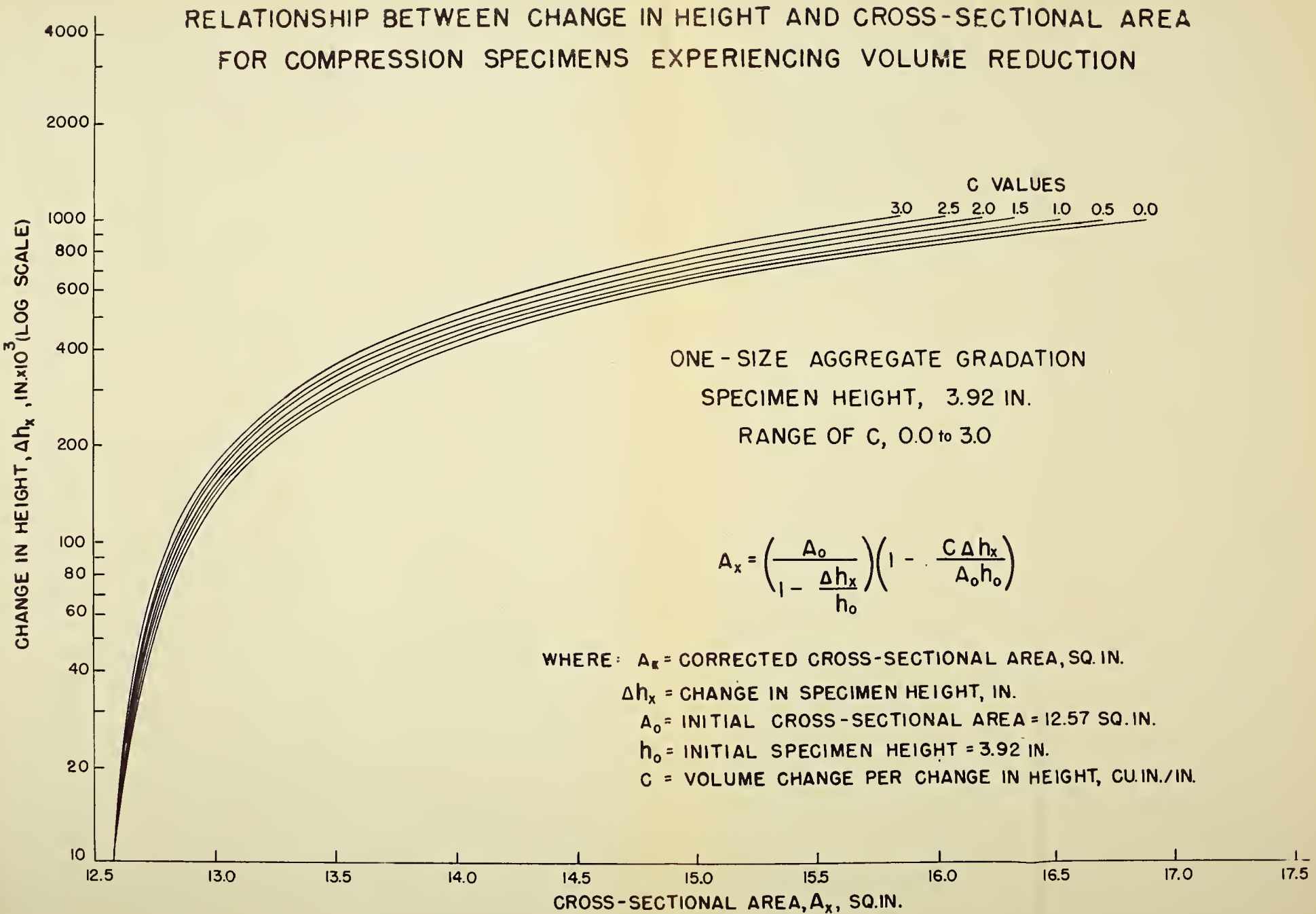
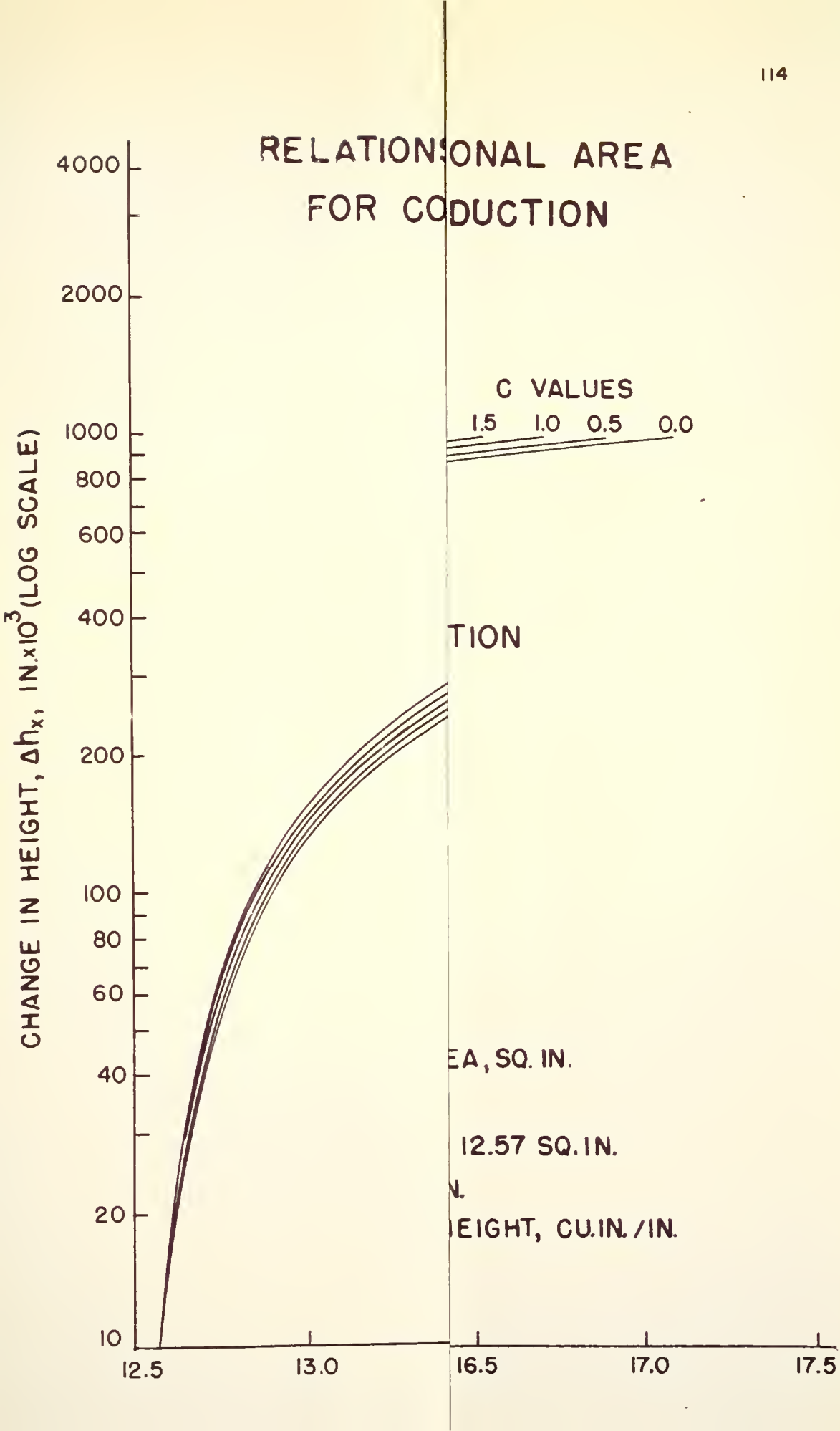

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA
FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION
C VALUES
3.0 2.5 2.0 1.5 1.0 0.5 0.0
ONE-SIZE AGGREGATE GRADATION
SPECIMEN HEIGHT, 3.92 IN.
RANGE OF C, 0.0 to 3.0
$A_x = \left(\frac{A_o}{1 - \frac{\Delta h_x}{h_o}}\right)\left(1 - \frac{C \Delta h_x}{A_o h_o}\right)$
WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA, SQ. IN.
Δh_x = CHANGE IN SPECIMEN HEIGHT, IN.
A_o = INITIAL CROSS-SECTIONAL AREA = 12.57 SQ. IN.
h_o = INITIAL SPECIMEN HEIGHT = 3.92 IN.
C = VOLUME CHANGE PER CHANGE IN HEIGHT, CU. IN./IN.
CHANGE IN HEIGHT, Δh_x, IN. x 10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
13.0
13.5
14.0
14.5
15.0
15.5
16.0
16.5
17.0
17.5
CROSS-SECTIONAL AREA, A_x, SQ. IN.

FIG. 38


RELATIONSONAL AREA
FOR CODUCTION
C VALUES
1.5 1.0 0.5 0.0
TION
EA, SQ. IN.
12.57 SQ. IN.
N.
IEIGHT, CU.IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.x10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
13.0
16.5
17.0
17.5

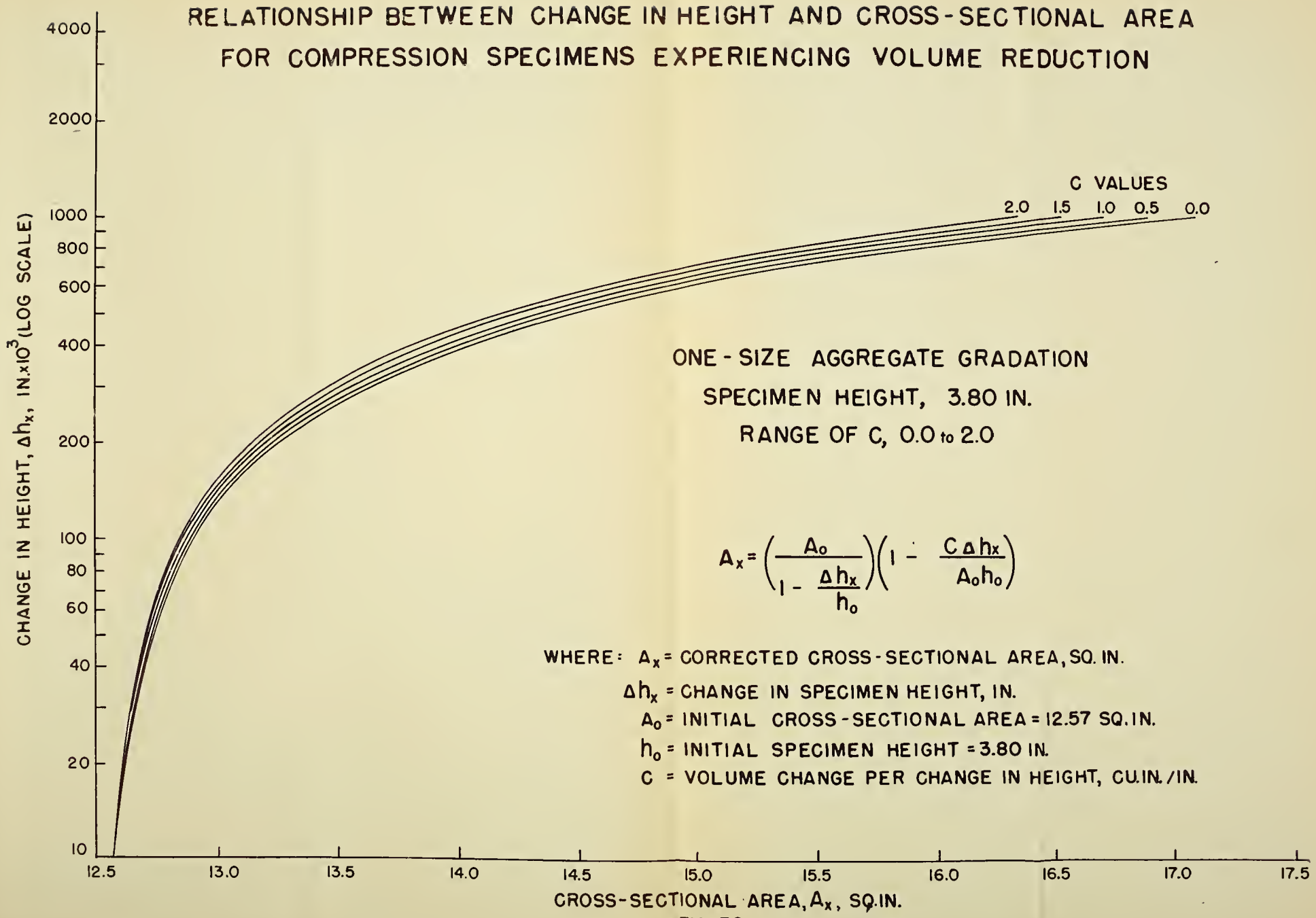

RELATIONSHIP BETWEEN CHANGE IN HEIGHT AND CROSS-SECTIONAL AREA
FOR COMPRESSION SPECIMENS EXPERIENCING VOLUME REDUCTION
C VALUES
2.0 1.5 1.0 0.5 0.0
ONE-SIZE AGGREGATE GRADATION
SPECIMEN HEIGHT, 3.80 IN.
RANGE OF C, 0.0 to 2.0
$A_x = \left(\frac{A_o}{1 - \frac{\Delta h_x}{h_o}}\right)\left(1 - \frac{C \Delta h_x}{A_o h_o}\right)$
WHERE: A_x = CORRECTED CROSS-SECTIONAL AREA, SQ. IN.
Δh_x = CHANGE IN SPECIMEN HEIGHT, IN.
A_o = INITIAL CROSS-SECTIONAL AREA = 12.57 SQ. IN.
h_o = INITIAL SPECIMEN HEIGHT = 3.80 IN.
C = VOLUME CHANGE PER CHANGE IN HEIGHT, CU. IN./IN.
CHANGE IN HEIGHT, Δh_x, IN.x10^3 (LOG SCALE)
4000
2000
1000
800
600
400
200
100
80
60
40
20
10
12.5
13.0
13.5
14.0
14.5
15.0
15.5
16.0
16.5
17.0
17.5
CROSS-SECTIONAL AREA, A_x, SQ. IN.

FIG. 39

APPENDIX C

TRIAXIAL TEST DATA

The triaxial compression test data which have been obtained in this laboratory investigation are presented in tabular form in this appendix, Tables 6 to 15. The asphalt content is expressed as the percent by weight of the total bituminous mixture.

TABLE 6

Triaxial Test Results — Open-Graded Mixture

Crushed Limestone — 10-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi			Density lb/cu ft
	1.0% Strain	5.0% Strain	Peak	
0	---	---	164	148.2
0	---	---	168	146.8
0 (avg)	---	---	166	147.5
30	387	---	396	146.7
30	377	---	384	146.7
30 (avg)	382	---	390	146.7
60	431	---	445	147.4
60	434	---	455	147.3
60 (avg)	433	---	450	147.4
90	498	---	535	147.7
90	483	---	533	149.2
90 (avg)	491	---	534	148.5
120	492	586	590	146.7
120	512	594	594	146.3
120 (avg)	502	590	592	146.5
150	592	734	739	145.9
150	571	719	726	145.2
150 (avg)	582	727	733	145.6

TABLE 7

Triaxial Test Results — Open-Graded Mixture

Crushed Limestone — 4-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress psi	Density lb/cu ft
0	286	148.1
30	404	149.2
60	488	143.2
90	626	142.9
120	765	143.0
150	840	142.8

TABLE 8

Triaxial Test Results — Open-Graded Mixture

Crushed Limestone — 10-in. High Specimen

Confining Pressure psi	4.0% Asphalt		5.0% Asphalt		6.0% Asphalt	
	Normal Stress psi	Density lb/cu ft	Normal Stress psi	Density lb/cu ft	Normal Stress psi	Density lb/cu ft
30	245	144.0	390	146.7	211	147.6
90	500	145.0	534	148.5	---	---
150	722	143.9	733	145.6	669	147.7

TABLE 9

Triaxial Test Results .. Open-Graded Mixture

Uncrushed Gravel and Natural Sand 10-in. High Specimen

Confining Pressure psi	4.0% Asphalt		5.0% Asphalt		6.0% Asphalt	
	Normal Stress psi	Density lb/cu ft	Normal Stress psi	Density lb/cu ft	Normal Stress psi	Density lb/cu ft
30	155	142.0	179	143.3	158	143.1
90	400	140.0	416	141.6	---	---
150	667	142.1	660	145.2	530	147.4

TABLE 10

Triaxial Test Results — One-Size Mixture

Crushed Limestone — 10-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
0	10	---	---	---	113.2	113.2	0.0	0.0
0	17	---	---	---	115.0	115.0	0.0	0.0
0 (avg)	14	---	---	---	114.1	114.1	0.0	0.0
30	99	141	154	158	112.9	113.4	0.5	1.20
30	106	143	154	156	111.7	112.2	0.5	1.26
30 (avg)	103	142	154	157	112.3	112.8	0.5	1.23
60	177	233	250	254	113.6	114.1	0.5	1.07
60	170	233	254	262	113.4	114.3	0.9	1.86
60 (avg)	174	233	252	258	113.5	114.2	0.7	1.47
90	226	282	314	324	113.8	115.2	1.4	0.98
90	223	274	290	332	113.4	114.2	0.8	0.60
90 (avg)	225	278	302	328	113.6	114.7	1.1	0.79
120	240	344	400	419	116.0	121.2	5.2	2.76
120	245	324	383	421	113.3	116.2	2.9	2.19
120 (avg)	243	334	392	420	114.7	118.7	4.0	2.48
150	296	429	511	540	115.3	117.2	1.9	1.46
150	310	414	497	536	115.1	120.9	5.8	6.10
150 (avg)	303	422	504	538	115.2	119.1	3.9	3.78

* C - change in volume per change in height

TABLE 11

Triaxial Test Results — One-Size Mixture (Vibrated)

Crushed Limestone — 10-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
30	88	119	131	136	115.5	116.8	1.3	1.02
60	160	213	231	235	115.6	115.6	0.0	0.01
90	194	282	330	357	118.6	119.0	0.4	0.47
120	259	359	420	464	117.3	121.1	3.8	3.68
150	325	448	523	557	116.7	119.2	2.5	2.22

* C - change in volume per change in height

TABLE 12

Triaxial Test Results — One-Size Mixture

Crushed Limestone — 4-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
0	15	---	---	---	111.8	111.8	0.0	0.0
30	73	113	125	133	114.6	114.7	0.1	0.21
60	126	189	225	240	110.2	113.1	2.9	2.25
90	180	268	324	369	110.1	115.1	5.0	2.57
120	250	366	449	498	112.2	116.9	4.7	3.40
150	288	436	524	590	111.9	113.4	1.5	1.57

* C - change in volume per change in height

TABLE 13

Triaxial Test Results — One-Size Mixture (Vibrated)

Crushed Limestone — 4-in. High Specimen

Asphalt Content - 5.0%

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
30	56	158	190	198	113.9	117.1	3.2	1.26
60	128	242	282	296	116.3	118.3	2.0	1.22
90	176	329	380	415	114.8	115.9	1.1	0.53
120	200	366	453	509	115.0	117.1	2.1	1.33
150	288	473	563	641	117.0	118.7	1.7	1.42

* C - change in volume per change in height

TABLE 14

Triaxial Test Results — One-Size Mixture

Crushed Limestone — 10-in. High Specimen

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
Asphalt Content - 5.0%								
30	103	142	154	157	112.3	112.8	0.5	1.23
90	225	278	302	328	113.6	114.7	1.1	0.79
150	303	422	504	538	115.2	119.1	3.9	3.78
Asphalt Content - 4.0%								
30	76	106	122	127	112.8	114.4	1.6	1.20
90	192	266	302	323	108.8	111.6	2.8	2.19
150	302	416	493	532	111.6	117.7	6.1	4.30
Asphalt Content - 3.0%								
30	78	105	118	126	111.1	113.9	2.8	2.11
150	289	391	469	520	110.4	115.0	4.6	4.06

* C - change in volume per change in height

TABLE 15

Triaxial Test Results

Uncrushed Gravel and Natural Sand

One-Size Mixture

10-in. High Specimen

Confining Pressure psi	Normal Stress - psi				Density - lb/cu ft			C* sq in.
	1.0% Strain	5.0% Strain	10.0% Strain	15.0% Strain	Before Test	After Test	Increase	
Asphalt Content - 5.0%								
30	56	79	83	84	111.9	112.3	0.4	0.23
90	167	232	258	282	113.6	114.2	0.6	0.52
150	262	374	423	441	113.0	114.4	1.4	1.17
Asphalt Content - 4.0%								
30	58	72	75	75	112.7	113.6	0.9	0.16
90	165	222	246	256	112.1	113.1	1.0	2.46
150	247	332	379	406	113.2	114.5	1.3	1.01
Asphalt Content - 3.0%								
30	56	72	79	85	113.0	114.3	1.3	0.89
150	262	340	378	400	113.1	114.6	1.5	1.38

* C - change in volume per change in height

APPENDIX D

HVEEM STABILOMETER TEST

Since the Hveem Stabilometer test is recognized as a modified triaxial test of the closed-system type, open-graded and one-size specimens were tested in the Stabilometer. These specimens were molded with both crushed limestone aggregates and uncrushed gravel and natural sand aggregates at the various asphalt contents as given in Table 4. This permitted a comparison of the results using the Stabilometer with the data from the triaxial test procedure for the given materials.

The methods utilized in conducting these tests were those outlined in the publication of The Asphalt Institute, Mix Design Methods for Hot-Mix Asphalt Paving (47), except for the following discrepancy. The specimens were formed by the double-plunger compaction procedure rather than by means of the mechanical kneading compactor. A description of the double-plunger compaction method is outlined in the section, Apparatus and Procedure, of this report and under the heading, Preparation of the Test Specimen. The Hveem Stabilometer is shown during a test in Fig. 40. A sample data sheet is presented in Fig. 41. The Stabilometer test results are given in Tables 16 to 19, where the asphalt content is expressed as the percent by weight of the total bituminous-aggregate mixture.

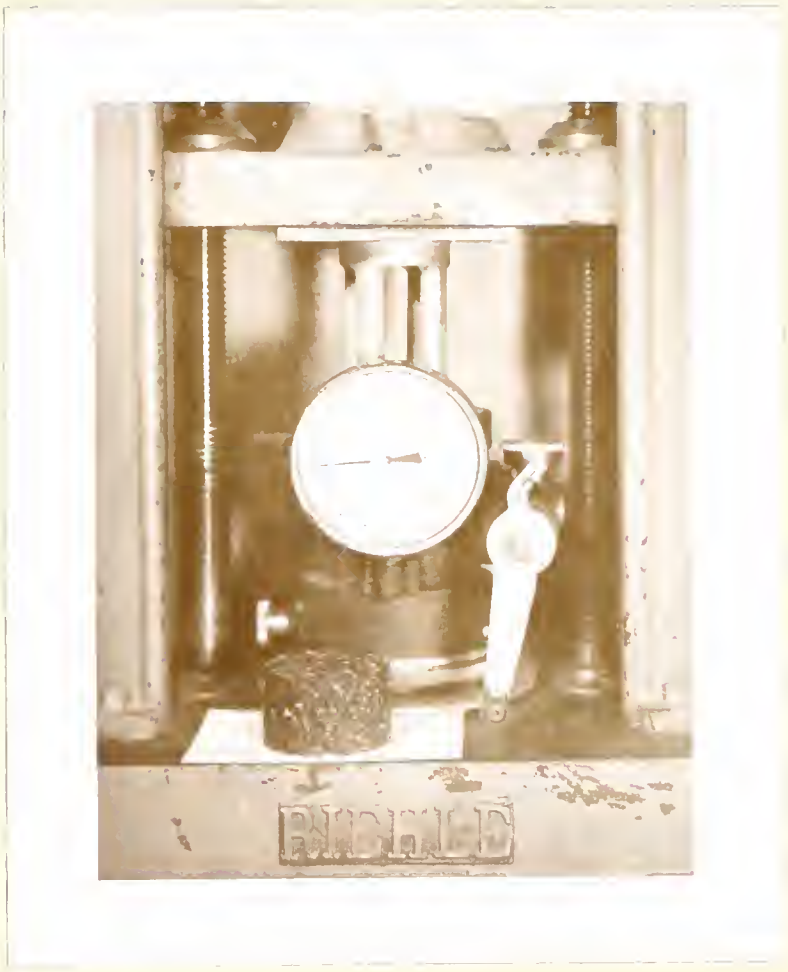


Fig. 40 Hveem Stabilometer Test

DATA SHEET

Stabilometer Test - Bituminous Mixtures

Specimen No. 0-1-5 Date Tested JULY 24, 1956

Initial Ht 2.50 in. By J.O. & R.P.

Weight 1200.0 gm Temperature 140F

Initial Dia 4.00 in. Unit Wt 145.5 lb/cuft

Load	P_v	P_h
1000		9
2000		12
3000		18
4000		27
5000	400	42
6000		62

No. of Turns to Increase P_h from 5 psi to 100 psi 3.30

Fig. 41

TABLE 16

Stabilometer Test Results Open-Graded Mixture

Crushed Limestone

Percent Asphalt	Stability	Density lb/cu ft
4.0	34	140.2
4.0	35	141.8
4.0 (avg)	35	141.0
5.0	36	145.5
5.0	38	143.9
5.0 (avg)	37	144.7
6.0	30	144.5
6.0	27	145.7
6.0 (avg)	29	145.1

TABLE 17

Stabilometer Test Results Open-Graded Mixture

Uncrushed Gravel and Natural Sand

Percent Asphalt	Stability	Density lb/cu ft
4.0	27	138.5
4.0	17	136.6
4.0 (avg)	22	137.6
5.0	25	140.6
5.0	23	141.0
5.0 (avg)	24	140.8
6.0	18	143.0
6.0	25	139.8
6.0 (avg)	22	141.4

TABLE 18

Stabilometer Test Results One-Size Mixture

Crushed Limestone

Percent Asphalt	Stability	Density lb/cu ft
3.0	9	110.6
3.0	9	105.9
3.0 (avg)	9	108.3
4.0	8	110.1
4.0	9	106.7
4.0 (avg)	9	108.4
5.0	7	111.9
5.0	8	109.2
5.0 (avg)	8	110.6

TABLE 19

Stabilometer Test Results One-Size Mixture

Uncrushed Gravel and Natural Sand

Percent Asphalt	Stability	Density lb/cu ft
3.0	7	110.1
3.0	9	112.1
3.0 (avg)	8	111.1
4.0	6	109.4
4.0	7	107.8
4.0 (avg)	7	108.6
5.0	6	109.0
5.0	6	108.3
5.0 (avg)	6	108.7